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CALTRAIN SAN FRANCISCO  
DOWNTOWN EXTENSION PROJECT  
CONCEPTUAL DESIGN AND DRAFT EIS/EIR

# Tunnel Engineering Analysis and Cost Report

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# CALTRAIN SAN FRANCISCO DOWNTOWN EXTENSION DEIS/DEIR

## TUNNEL ENGINEERING AND COST ESTIMATE REPORT

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PREPARED FOR THE

CORRIDOR JOINT POWERS BOARD

PREPARED BY  
DAMES & MOORE FOR  
ICF KAISER ENGINEERS, INC.

June 14, 1996

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CalTrain San Francisco  
downtown extension  
1996.

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June 14, 1996  
Job No. 30751-005-003

ICF Kaiser Engineers, Inc.  
1800 Harrison Street  
Oakland, CA 94612-3430

Attention: Mr. David Minister  
Senior Project Manager

**Report**  
**CalTrain San Francisco Downtown Extension**  
**Tunnel Engineering and Cost Estimate**

We are pleased to present the results of the Tunnel Engineering and Cost Estimate study. This study was undertaken at the request of ICF Kaiser Engineers, Inc., and is in accordance with our proposal dated January 4, 1996, and Amendment No. 3 of our Contract No. 65928-SC-03 with ICF Kaiser Engineers, Inc.

A draft of this report was issued on May 21, 1996 for review and comments by other team members and by Professor Thomas D. O'Rourke, the Chairman of the Board of Special Consultants. A revised draft incorporating comments and suggested changes from the reviewers was resubmitted on June 7, 1996. The comments provided by the various reviewers have been considered carefully and the report modified as required to address the review comments.

The report focuses on the development of safe tunneling methods, ground improvement, and underpinning methods to protect existing structures from the effects of tunneling. It consists of the main text prepared by Dames & Moore and four appendices. Appendix A presents review comments from Professor Thomas D. O'Rourke. Appendix B is the report prepared by Haley & Aldrich and addresses tunneling methods. Appendix C is the report prepared by Nicholson Construction Company, the specialists for ground improvement and support systems. Appendix D is the report prepared by Werner and Associates and addresses protection of existing structures by underpinning methods.

If you have any questions regarding the contents of this report, we would be pleased to discuss them with you.

Very truly yours,

DAMES & MOORE

D. Koutsoftas  
Principal



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# **CALTRAIN SAN FRANCISCO DOWNTOWN EXTENSION PROJECT TUNNEL ENGINEERING AND COST ESTIMATE**

## **1.0 INTRODUCTION**

This report summarizes the results of a series of specialized studies conducted to evaluate tunneling techniques, ground reinforcement methods, and underpinning methods, and their feasibility and cost effectiveness toward the specific purpose of controlling the impacts of tunnel construction on historic structures and other facilities along the portion of Alternative 3 that extends from Third and Townsend Streets to the intersection of Bryant and Colin P. Kelly, Jr. Streets.

This study is part of the EIS/EIR effort for the CalTrain San Francisco Downtown Extension project. As part of the EIS/EIR studies for the CalTrain project, Dames & Moore previously completed a series of geotechnical studies to evaluate the feasibility of various alternative underground CalTrain subway alignments. The results of Dames & Moore's previous studies were presented in the following two reports:

1. Geotechnical Site Investigation, dated September 25, 1995; and
2. Geotechnical Engineering Recommendations, dated December 27, 1995.

In addition to Dames & Moore's investigations, a Board of Geotechnical Consultants was convened to review the Site Investigation and Geotechnical Recommendations. Reports prepared by the Board of Consultants are included as Appendices to the Geotechnical Engineering Recommendations report.

Based on the results of the previous studies and other relevant community and environmental considerations, the alignment shown on Plate 1 was chosen as the locally preferred alternative. In the east-west direction, the alignment runs along Townsend Street with the southern tunnel portal just west of Fourth Street. The north-south alignment, runs along Colin P. Kelly, Jr. and Essex streets, with the northern tunnel portal just north of Folsom Street. The section north of Folsom Street runs in a cut-and-cover subway box under an extension of Essex Street and then turns eastward between Minna and Natoma Streets into the underground Transbay Terminal site.

The east-west portion of this alignment along Townsend Street and the north-south portion along Colin P. Kelly, Jr. are connected by a curved portion, as shown on Plate 1. Three different curved routes are being considered as shown on Plate 2, and as described below.



1. A **“long radius”** tunnel that begins to curve away from Townsend near Third Street and connects to the north-south portion of the alignment near Bryant Street. The total length of the tunnel for this option (between stations 169+00 and 191+15, and from 194+75 to 208+00) is 3,540 feet long.
2. A **“medium radius”** tunnel that diverges from Townsend Street near the intersection with Stanford Street, and connects to the Colin P. Kelly, Jr. alignment near Brannan Street. The total length of tunnel for this option (between stations 169+00 and 186+55 and from 188+30 to 208+00) is 3,725 feet long.
3. A **“short radius”** tunnel that diverges from Townsend Street near the intersection with Second Street and runs parallel to Colin P. Kelly, Jr. Street, approximately 300 feet north of Townsend Street. The total length of tunnel for this option (from station 169+00 to 208+00) is 3,900 feet.

All three curved alignment options pass under a number of historic structures between Third and Bryant Streets. The protection of these structures from potential damage that could be caused by the ground movements caused by tunneling is a major concern for the CalTrain project. The objective of the current study is to investigate methods of tunneling, ground improvement, and building supports that would allow construction of the proposed train tunnels with minimal impact on structures that are within the zone of influence of the tunneling operations.

As part of this study, Dames & Moore engaged the services of three specialist subconsultants to assist in evaluating the potential impacts of tunneling under these historic buildings and to develop mitigation measures to control such impacts within acceptable tolerances. Each of these subconsultants prepared a separate report addressing the specific tasks assigned to them. Their reports are attached to this report as appendices B, C, and D. The remainder of this report consists of a brief review of the purpose and scope of Dames & Moore's investigation, followed by a review of the tunnel alternatives. Section 3.0 discusses geotechnical considerations and potential impacts of tunneling on existing structures and facilities. Section 4.0 discusses tunneling and mitigation measures, and Section 5.0 provides a summary and recommendations.

## 2.0 PURPOSE AND SCOPE

As shown on Plate 2, the three curved alignments under consideration would pass under a variety of buildings. AGS, Inc., has prepared a thorough survey of these buildings and their findings are presented elsewhere as part of the reports prepared by AGS, Inc. Many of these buildings are



very old, one- or two-story unreinforced brick masonry structures supported on shallow foundations. Some of these structures have experienced the effects of previous earthquakes, and may have been weakened by settlements caused by earthquakes or other causes.

In view of the historic nature and sensitivity to settlement of these buildings, the Joint Powers Board decided to investigate methods that would be employed to minimize the impacts of tunneling and to mitigate potential adverse impacts of tunneling. The goal of this investigation is to develop construction techniques that will protect existing buildings from damage caused by ground movements induced by tunneling. To achieve this objective Dames & Moore undertook the following scope of services:

### **Task No. 1 Engineering Services**

This task includes evaluation of different tunneling methods, ground treatment, and strengthening or underpinning existing structures to minimize the effects of the proposed tunnel construction on existing buildings, roadways, and utility lines. The following subtasks were included:

#### **1.1 Evaluation of Tunneling Methods**

Haley & Aldrich, Inc., a firm with specialist capabilities in tunnel design and construction methods, was engaged to provide an independent evaluation of potential tunneling methods that could be used to construct the proposed tunnel(s). The study focused on various alternatives that would minimize potential adverse effects on existing structures, evaluation of risks and their associated costs, and potential mitigation measures. The study report prepared by Haley & Aldrich is included as Appendix B to this report.

#### **1.2 Evaluation of Ground Treatment Methods**

Nicholson Construction Company, a firm specializing in underground construction, ground treatment, and ground reinforcement techniques, was engaged to review the existing subsurface conditions and develop suitable methods for ground treatment and strengthening, so that the effects of tunneling on existing structures could be maintained within tolerable limits. Nicholson's report summarizing their findings and recommendations is included as Appendix C.



### **1.3 Evaluation of Building Strengthening Methods**

Werner & Associates, a firm specializing in shoring design, underpinning, and underground construction, was engaged to evaluate methods to strengthen the buildings and/or improve their foundation support so that they would be unaffected by the proposed tunneling. Werner's report presenting their findings and recommendations is included as Appendix D.

### **1.4 Review of Tunneling Costs**

Because of the very significant costs associated with tunneling, it was considered necessary to provide an independent assessment of the costs associated with the various tunneling methods considered feasible for the CalTrain project. To achieve this objective, Haley & Aldrich, Inc., was engaged to carry out this task. Their report, which is combined with their report under subtask No. 1, is included in Appendix B to this report.

### **1.5 Coordination and Technical Support by Dames & Moore**

The work associated with the various subtasks described above was coordinated and managed by Dames & Moore.

In addition to coordination and management, Dames & Moore met on several occasions with the subconsultants to brief them on the goals and objectives of the project, to review ground conditions and other technical issues, and to supply available geotechnical data and other relevant technical documents. During the course of the project, we provided guidance and direction to the specialist subconsultants, reviewed the progress of the work and drafts of their reports, provided comments and suggestions, and coordinated input to the study from other team members. We also arranged and coordinated an independent review of the various drafts prepared by the specialists, as well as of this report, by Professor Thomas D. O'Rourke of Cornell University, the Chairman of the Board of Consultants for the CalTrain project. His comments are summarized in his letter report, which is attached to this report as Appendix A. Finally, we prepared this report, which summarizes the findings and recommendations of the study.

## **3.0 GEOTECHNICAL CONSIDERATIONS**

The general geotechnical concerns regarding tunneling have been discussed at length in "CalTrain Downtown San Francisco Extension DEIS/DEIR: Geotechnical Engineering Recommendations dated December 27, 1995," prepared by Dames & Moore, and the reports



prepared by the Board of Consultants. The primary technical concerns with tunneling are summarized below:

1. As the previous reports indicated, tunneling is technically feasible, and a safe method for constructing the underground portion of the CalTrain Downtown Extension Project. However, it must be recognized that the tunnels will have to be excavated through extremely poor rock, below the groundwater table, and with relatively shallow ground cover. This is very difficult, risky and costly. Managing the risks of potential cave-ins and/or excessive ground movements and their impacts on existing structures and services is a primary consideration. Therefore, the focus of this study is to develop construction methods that specifically address and mitigate risks associated with tunneling.
2. The tunneling will have to be performed under mixed ground conditions, because of the high variability and the high degree of weathering and fracturing of the rock. The tunneling scheme initially developed in consultation with the Board of Consultants involved incremental excavation by hand mining techniques intended to secure the stability of the face and roof of the tunnels. This was considered to be conservative and safe, but also a time consuming and expensive process. Even with this method, there was considerable concern with the potential for caving because of the damage such an occurrence would cause. Recent tunnel failures at the Los Angeles Metro, Athens Metro (Greece), and the Heathrow Airport tunnels in England confirmed the perception that accidental cave-in is a serious concern. Because the CalTrain tunnels would pass under a large number of buildings, it is considered essential that the tunneling methods used reduce the potential for cave-in to an extremely low probability. One of the objectives of the current study is to develop procedures that increase the degree of confidence that the tunnel excavation can be completed safely with minimal effects on existing structures. Safe excavation is intended to mean that the potential of cave-in is virtually eliminated.
3. The Board of Consultants advised against the use of shield-driven tunnels, because of the high variability of the rock. The Board's primary concern was that in the event of a cave-in, the shield would be trapped in the ground. Such an incident would not only cause damage to existing structures and risk the safety of building occupants, but it would also cause very significant delays to the project and disruptions to traffic and services, in order to excavate around the shield equipment to free it. However, the recently completed Richmond Transport sewer tunnel for the San Francisco Clean



Water Program showed that shield-driven tunnels could be successfully constructed in Franciscan rock, under the right set of circumstances. Therefore, the firm of Haley & Aldrich, who acted as the construction manager for the Richmond Transport tunnels, was asked to reevaluate the feasibility and safety of shield-driven tunnels for the CalTrain project.

4. The Board of Consultants recommended that the CalTrain tunnel profiles be made as deep as possible so that the thickness of the rock cover above the tunnel crown (top) was as great as possible. Plates 3 and 4 show the new tunnel profiles, with approximate top of rock, that were developed in response to the Board's recommendations. The amount of available rock cover varies substantially over the length of the alignment. The minimum rock cover is estimated at about 15 to 20 feet. The amount of estimated rock cover is based on limited borehole information, and may be less than that estimated at some locations. Zones of local concentrations of rock weathering may result in soil-like material extending below the proposed tunnel crown and locations of ancient streams and drainage channels may be filled with soils locally deeper than implied by the average top of rock shown on Plates 3 and 4. The overall objective of the tunnel profiles shown on Plates 3 and 4 was to maintain the total (soil and rock) cover above the tunnel crown greater than 30 feet.
5. In order to construct the tunnels safely and efficiently, the ground must be dewatered. Groundwater levels in the area of interest are relatively shallow, ranging from 10 to 20 feet below existing grade. Typical groundwater levels are expected to be above the crown (top) of the tunnels. As pointed out in Dames & Moore's "Geotechnical Recommendations" report, because of the very poor condition of the rock and because of the likelihood that soils mixed with rock may be encountered, the groundwater levels will have to be lowered below the tunnel invert (bottom) to improve the stability of the excavation and to increase production rates.

It must be recognized that the three curved tunnel alignments (shown on Plate 2) pass under private property and there is little suitable access for installation of dewatering wells. It should be assumed that the property owners would not provide access to their properties to install dewatering wells. In this case, the spacing of dewatering wells along any given alignment could be 200 feet or more. Clearly such a spacing would not be sufficient to achieve the required drawdowns. Of the three curved alignments, the short radius alignment has the greatest advantage (in terms of



dewatering) because wells can be installed along most of the alignment along Colin P. Kelly, Jr. and Townsend streets in public street right-of-ways.

In areas where dewatering cannot be performed from the surface, the dewatering will have to be carried out from inside the tunnel. The construction of drifts in advance of the main tunnel excavation will partially fulfill this function. However, because the drifts are planned to be high above the invert of the tunnel (top heading), additional dewatering would be required before the bench excavation is undertaken.

Dewatering would be problematic for shield-driven tunnels because this method will not have the benefit of the top drifts. Therefore, more extensive dewatering would be required, should the shield driven tunnel construction technique be used.

#### **4.0 TUNNELING AND MITIGATION MEASURES**

This chapter summarizes the main points of the findings from the three specialist consulting firms. Each of the specialist firms was asked to develop recommendations to minimize the impact of CalTrain tunnel construction on structures within the tunnel zone of influence.

Three basic techniques were examined: ground treatment, tunneling methods, and underpinning. Each of these techniques is described and recommendations are summarized in the following sections.

##### **4.1 Ground Treatment**

The basic problem and concern with tunneling for the CalTrain project is the poor condition of the rock through which the tunnel would be built, and the possibility of encountering soil-like material in the tunnel face. All aspects of tunnel construction, including the stability of the tunnel face and roof, the required temporary supports, production rates, and the permanent lining are affected by the quality of the rock, the presence of soil, and groundwater conditions. The objective of ground improvement is to strengthen the rock so that the stability of the opening can be improved and the effects of tunneling on existing structures can be controlled within tolerable limits.

Nicholson Construction Company, a specialist on ground treatment, reviewed the available geotechnical information, inspected the rock cores, and prepared an assessment of treatment methods that could be used to improve the ground through which the tunnel(s) will be excavated. Results of this assessment are summarized below.



Nicholson considered two treatment processes that are commonly used in rock: permeation-type grouting and ground reinforcement. Permeation type grouting consists of drilling grout holes either from the ground surface above the tunnels, or from the face of the tunnel and pumping a fluid, cement-based grout into the ground around the proposed tunnel. The grout fills cracks and other discontinuities and under the right circumstances might penetrate the weathered rock, thus cementing the rock and sediments together to strengthen the area through which the tunnel will be built. Ground reinforcement involves drilling small diameter holes through the face of the tunnel, installing steel reinforcing elements, and grouting them in place to form a stronger, stiffer and less variable material.

Nicholson's assessment is that the rock along the proposed CalTrain tunnel alignment is highly variable, containing significant amounts of weathered material. It is expected that a considerable portion of the weathered materials may be clayey in nature. Nicholson concluded that under these conditions, permeation-type grouting would not be effective to increase the strength of the ground sufficiently so as to safeguard against the risks of instability of the face and excessive ground movements.

Nicholson also concluded that the only reliable means to improve uniformly and positively the types of rock found along the CalTrain alignment would be by ground reinforcement. The proposed method, called forepoling or spiling, is illustrated on Plates 5 and 6. This method consists of installing steel pipes in predrilled holes along the roof and side walls of the tunnel, in advance of excavating the tunnel, creating an umbrella arch over the tunnel. The arch is supported with steel ribs and/or shotcrete as excavation progresses. At any one time only a short segment, several feet long, is exposed between the tunnel face and the last support. The stability of the unsupported portion of the tunnel is safeguarded by the steel reinforcing elements, which act as short beams supported at the one end in the unexcavated rock and at the other end by the steel ribs and shotcrete. The reinforcing elements are overlapped for increased stability. The length of overlap can be varied depending on the nature of the ground encountered. In areas where the ground is particularly weak and where the tunnel passes under settlement-sensitive structures, the overlap can be up to 20 feet, as shown on Plate 6. More details about this method are provided in Nicholson's report, Appendix C.

Spiling constructed in this manner has been used primarily in Europe and Japan. Several technical papers describing the application of this technique for tunneling in weak ground, with the objective of protecting existing structures, are included in the Nicholson report. A recent paper by Murata and Okazawa (1996) describes the application of this technique for a tunnel project in Kobe City, Japan. The Japanese tunnels were horseshoe-type, 54 feet wide and 36 feet tall. Overburden cover ranged from 23 feet to 100 feet. The sequence of reinforcement and tunnel excavation is very similar to the method proposed by Nicholson for the CalTrain tunnel. Settlements measured during



construction of the Kobe City tunnels were less than 1 inch, even though the ground, consisting primarily of sands and gravels below the water table, was very susceptible to tunneling disturbance. It is noteworthy that this tunnel was under construction during the magnitude 6.9 Kobe earthquake of January 17, 1995. The tunnel survived the earthquake without damage. Several applications of a variation of this technique in Europe are described in the April 1996 issue of Ground Engineering magazine.

Information on production rates and costs for the installation of the reinforcements are provided in the Nicholson report (Appendix C).

Implementation of the Nicholson proposal for reinforcement will require, for reasons of efficiency, construction along two mining fronts. Essentially, the process consists of two operations: ground reinforcement and excavation. Ground reinforcement is undertaken along the one front, while mining is taking place at the other front. When excavation within the previously reinforced ground at one front is complete, the two operations switch places. This process will require a tunnel shaft near the middle of the tunnel alignment.

## **4.2 Tunneling Methods**

Haley & Aldrich, Inc. was engaged to evaluate tunneling methods and provide recommendations for methods of tunneling that could control impacts on existing structures within tolerable limits and to estimate tunnel production rates and overall tunneling costs. Haley & Aldrich tunnel engineers reviewed available geotechnical reports, inspected rock cores taken in the area of the project alignment, and visited the project site to develop an appreciation of the issues relative to protection of existing historic buildings along the proposed alignments. Haley & Aldrich's major findings and recommendations are summarized below.

1. The rock along the CalTrain tunnel alignment is of very poor to extremely poor quality, and tunneling was characterized over most of the alignment to be "Difficult to Hazardous."
2. Tunneling is technically feasible and economically viable. Appropriate precautionary measures can be implemented to control ground movements caused by tunneling within limits that are considered tolerable to existing structures. On average, settlements at the ground surface can be controlled to less than 1 inch, with maximum settlements along the alignment less than 2 inches.



3. Tunneling can be carried out either as a single tunnel to be constructed by mining techniques or as twin bored tunnels to be constructed as shield-driven tunnels. In either case, extensive ground reinforcement measures will be required to secure the stability of the tunnels and to control ground deformations within tolerances.

The scheme proposed for the single tunnel is shown on Plate 7. It involves excavation initially of two wall drifts, installation of reinforcement and the dewatering system, followed by incremental (staged) excavation, as shown on Plate 7. The wall drifts function both as drainage galleries and provide exploratory data, in addition to providing access for installation of ground reinforcement and dewatering. Excavation is carried out using roadheaders supplemented by light blasting as required by the ground conditions encountered in the tunnel.

The scheme proposed for twin bored tunnels is shown on Plate 8. It involves excavation using roadheaders and light blasting as necessary from inside the shield. Segmented concrete lining was recommended for ground support.

4. Dewatering to lower the groundwater levels below the tunnel invert will be required. Because of limited access from the surface, most of the dewatering will have to be carried out from inside the tunnel opening(s). For the single tunnel, dewatering will be performed by drilling horizontal drainage pipes through the face of the tunnel as necessary, and by installing dewatering wells from the wall drifts (see Plate 7). For the twin bore tunnels, Haley & Aldrich recommends the use of directional drilling methods to construct an 18-inch-diameter horizontal drainage pipe along the tunnel alignment for dewatering purposes. The intent of the horizontal pipe is to predrain the ground within the tunnel opening. Directional drilling involves drilling of a small pilot hole, typically 5 to 8 inches in diameter, using remotely operated equipment to control the direction of drilling. After the pilot hole is completed it can be enlarged to the desired size using specialized equipment, called reamers. A pipe is finally inserted into the hole to complete the installation.
5. Haley & Aldrich, Inc. provided cost estimates for the two tunneling methods recommended. At the request of ICF Kaiser Engineers, Inc., Haley & Aldrich also prepared a cost estimate for the tunneling method proposed by Nicholson Construction Company, including the construction of a drift near the tunnel crown to provide exploratory information and to act as a drainage gallery. The average costs per lineal foot of tunnel for the three schemes are as follows:



- 1). Single tunnel with two wall drifts: \$14,000 per lineal foot.
- 2). Twin bores constructed as shield-driven tunnels: \$13,000 per lineal foot.
- 3). Single tunnel with spiling as proposed by Nicholson Construction and with a drainage/exploratory drift: \$14,000 per lineal foot.

The above estimates do not include the costs for shaft construction, final lining or muck disposal. The assumption for muck disposal was that some of the muck, being of good quality crushed rock, could be sold as fill, while the rest will have to be disposed of. This assumption may be proven right, but at this stage we would recommend that funds be allocated as a contingency in case some of the muck is found to be contaminated. Because of the high costs of disposal of contaminated soils, the costs of disposal could easily exceed the income from sale of good quality rock fill. The above costs also do not include the costs for Design, Project Management, Construction Management, Contingency, Project Reserve, etc.

#### **4.3 Provisions for Underpinning**

Underpinning is the process of strengthening or replacing the foundations of an existing structure to allow construction of new facilities, such as the CalTrain tunnel, within close proximity of the existing structure, without impacting adversely the existing structure.

Werner & Associates investigated feasible schemes for underpinning the existing historic buildings along the project alignment and provided rough cost estimates for each of the three curved tunnel alignments presently under consideration. Werner's report is attached as Appendix D to this report. The major findings and recommendations regarding underpinning are as follows:

1. The process of underpinning involves transfer of loads from the existing foundations to new foundations. This redistribution of loads will disturb the current equilibrium condition of the building, causing deflections that could result in some damage (cracking, etc.). Because the existing historic buildings are very old and settlement-sensitive structures, it will be difficult to implement any type of underpinning system without disturbing the buildings and potentially causing some damage, architectural or otherwise.
2. In view of the nature of the buildings involved, the best strategy appears to be employing proven, safe, and conservative tunneling methods that would virtually eliminate the risk of cave-ins and control ground movements within tight tolerances.



Any minor damage to the buildings that may develop would be repaired and the buildings restored to their preconstruction condition at the end of the project. This approach was successfully used for the MUNI Metro Turnback project.

3. Conventional underpinning schemes involving deep mined pits/piers, jacked piles, or pin piles are difficult, risky and very costly. Installation of any of these types of new foundations carries a significant risk that the underpinning process would cause damage to the buildings. The difficulties with underpinning for the CalTrain project are in part due to the large span over (the CalTrain tunnel) which the building structural loads will have to be transferred so that the new foundations are outside the zone of influence of tunneling.
4. A simpler and more economical underpinning, assuming that the risk associated with cave-ins due to tunnel collapse is managed through conservative tunneling techniques, would be to install a jacking system (passive underpinning) that allows periodic lifting of the buildings to compensate for settlements caused by tunneling. This is referred to in this report as "control type" underpinning. Inherent in this approach is a comprehensive monitoring program and development of threshold limits of settlement, which would trigger the lifting process. For details of this option please refer to Appendix D. However, it must be recognized that it would be necessary to be able to enter the buildings on short notice to carry out the jacking operations. This would, in some instances, inconvenience the building occupants.
5. If the intent of the underpinning work is to provide an extra measure of safety in the event that a cave-in occurs, then conventional underpinning by transferring the structure loads to new, deep foundations that derive their support from beyond the zone of influence of tunneling would be required. An example of this type of underpinning is shown on Plate 9. It must be recognized that the system shown on Plate 9 assumes that caving, if it were to occur, would be a localized incident, involving only a portion of the tunnel. If the intent was to safeguard against a case of major collapse involving the entire tunnel cavity, then a system much more robust than the one shown on Plate 9 would be required. However, in view of the ground reinforcement and incremental excavation methods planned for the CalTrain tunnels, the system shown on Plate 9 is considered a safe and satisfactory precaution.
6. The costs associated with underpinning vary considerably between the three proposed curved alignments. The lowest cost is associated with the "short radius" tunnel



because it involves constructing the tunnel beneath the fewest buildings. The highest cost is associated with the "long radius" tunnel because it would require tunnel construction beneath the largest number of buildings. Cost estimates range from \$0.6 million for the short radius alternative to \$1.5 million for the long radius alternative for "control type" underpinning, and from \$1.0 million (short radius) to \$2.5 million (long radius) for conventional underpinning.

7. The Werner & Associates report concludes that underpinning is not required north of Brannan Street because of the deeper soil and rock cover in this area. However, it must be recognized that the 355 Bryant Street building has been recently seismically upgraded, including rock anchors for uplift load resistance. Although at this time details of the retrofit design are not available, it must be recognized that special measures may be required to avoid interference with that building's existing foundation support system. It will be necessary to make an allowance in the cost estimate for potential redesign of the seismic uplift resistance of this building.

## **5.0 CONCLUSIONS AND RECOMMENDATIONS**

The results of the special studies for the CalTrain tunnels in the area of the historic buildings between Townsend and Bryant Streets leads to the following conclusions and recommendations:

1. Tunneling through highly weathered and fractured Franciscan rock, even with the new deeper profiles shown on Plates 3 and 4, is considered to be "difficult and hazardous" according to accepted tunnel classification systems, although perfectly feasible using proper construction techniques. In order to manage the risks associated with tunneling, a comprehensive system of ground improvement and dewatering, in combination with very careful tunneling methods and perhaps underpinning, will be required.
2. Three different tunneling methods have been identified as technically feasible, safe and economically attractive for constructing the proposed tunnels. All three methods will involve extensive ground strengthening to protect against potential cave-ins, and dewatering to control groundwater. The tunneling costs for the three methods identified in the study are very similar, ranging from \$13,000 to \$14,000 per lineal foot, but excluding any costs for final lining, muck disposal, and for underpinning, (if required).



3. Although the methods proposed are quite conservative and are intended to minimize to the lowest degree possible the risk of cave-ins, it is not possible to completely eliminate the risk of potential cave-ins. It is Dames & Moore's opinion that the risks associated with tunneling can best be managed by carefully selecting the contractor through a strict prequalification system, by comprehensive and vigilant engineering supervision, and by a careful monitoring program to evaluate the effectiveness of different techniques and procedures as construction progresses. A contractor prequalification process was used successfully for the MUNI Metro Turnback (MMT) project. A careful monitoring program with timely interpretation of the results and clearly defined action levels proved to be very successful in controlling construction processes on the MMT project, so that ground deformations could be kept within specified tolerances.

The risks are greatest for the "long radius" tunnel option, because it passes under the largest number of buildings, even though this alternative involves the shortest length of tunnel. The lowest risk, as far as the effects of tunneling on buildings are concerned, is for the "short radius" tunnel, because this option involves tunneling under the smallest number of buildings.

4. At this stage of the project, we recommend that underpinning be considered as an essential ingredient of the strategy for mitigation of the effects of tunneling on existing structures. A final decision on the need for building underpinning should be made during the preliminary design phase of the project after more investigations are performed to evaluate the effects of tunneling on the existing structures. Future evaluations must consider carefully the impact of the underpinning work on the occupants of each of the buildings. It is likely that in some instances buildings may have to be vacated for several months to allow construction of the underpinning system. The costs associated with business relocations are not included in the underpinning cost estimate and should be covered under contingency.
5. The "long radius" tunnel alternative will involve the most difficult dewatering, because access for dewatering from the surface is very limited. The "medium radius" tunnel option has somewhat better surface access for dewatering, while the "short radius" tunnel provides the best surface access for dewatering. Consequently, it should be expected that the costs associated with dewatering will be greatest for the "long radius" tunnel, because most of the dewatering will have to be carried out from inside the tunnel. Likewise, the least dewatering costs would be associated with the



“short radius” tunnel option. The lower dewatering costs and the lower costs for underpinning would offset partially the higher cost (approximately \$13 million) associated with the greater length (360 feet) of the “short radius” tunnel.

6. The “short radius” tunnel alignment passes under the empty lots at 251 and 257 Brannan Street. It is our understanding that a high-rise apartment building has been approved for the 251 Brannan lot. We recommend that the CalTrain project make provisions and also enter into negotiations with the owner of the new building so that the design of the new structure can accommodate the CalTrain tunnels without having to underpin the new structure. In effect, what is required is to incorporate the underpinning into the foundation design and construction plans for the new building up front.
7. It is Dames & Moore's opinion that the option involving twin bore shield-driven tunnels is the most risky of the three tunneling options considered. Working inside a shield does not provide as much flexibility in terms of excavation and treatment methods as working inside the bigger opening of the single tunnel. In addition, the need for reinforcing the rather narrow center pillar between the two bores seems to introduce an unnecessary operation that complicates the construction process. As mentioned earlier, the dewatering inside the smaller opening of the two bores will also be more difficult and expensive. But the most important consideration is the potential for significant slowdown in tunnel construction rates if there is a need to alternate frequently between roadheader excavation and blasting because of the high degree of variability of the rock. Finally, there is the concern that a cave-in, unlikely as it might seem, could trap the shield, causing significant delays in the work. In such a case, to free the shield, it may be necessary to excavate a shaft from the surface causing significant disruption to traffic and other facilities. Because of just such problems during construction of the Athens Metro, the use of shield excavation through poor ground conditions was discontinued (Civil Engineer International, July/August 1995).

In view of the concerns with shield-driven tunnels, we recommend that this option not be considered further, unless these issues are satisfactorily addressed in detail during the preliminary engineering phase of the project, and unless this option proves to be clearly superior in terms of safety, production rates, and costs. Professor O'Rourke is in agreement with this recommendation, as indicated in his review comments (Appendix A). At this stage of the analysis, the cost differential appears to be very



small and does not warrant the potential extra risks and difficulties associated with this option.

8. In view of the recent successes with the spiling (forepoling) method of ground improvement, and the apparent effectiveness of the method to improve the stability of the ground, we recommend that the option proposed by Nicholson Construction be selected as the preferred tunneling construction option. We would recommend, however, that the Nicholson option be modified to include a drift near the crown of the tunnel as shown on Plate 10. The objective of the drift is to provide partial drainage and to be used for exploratory purposes.
9. It must be recognized that for the option shown on Plate 10 to be most effective, TWO excavation fronts will be required. This means that the tunnel access shaft must be near the middle of the tunnel alignment. For the "short radius" and "medium radius" option, the shaft could be located in the parking lot at 280 Brannan Street. However, for the "long radius" tunnel, the shaft would have to be located in Brannan Street, between Second and Colin B. Kelly Jr. Streets (see Plate 2).



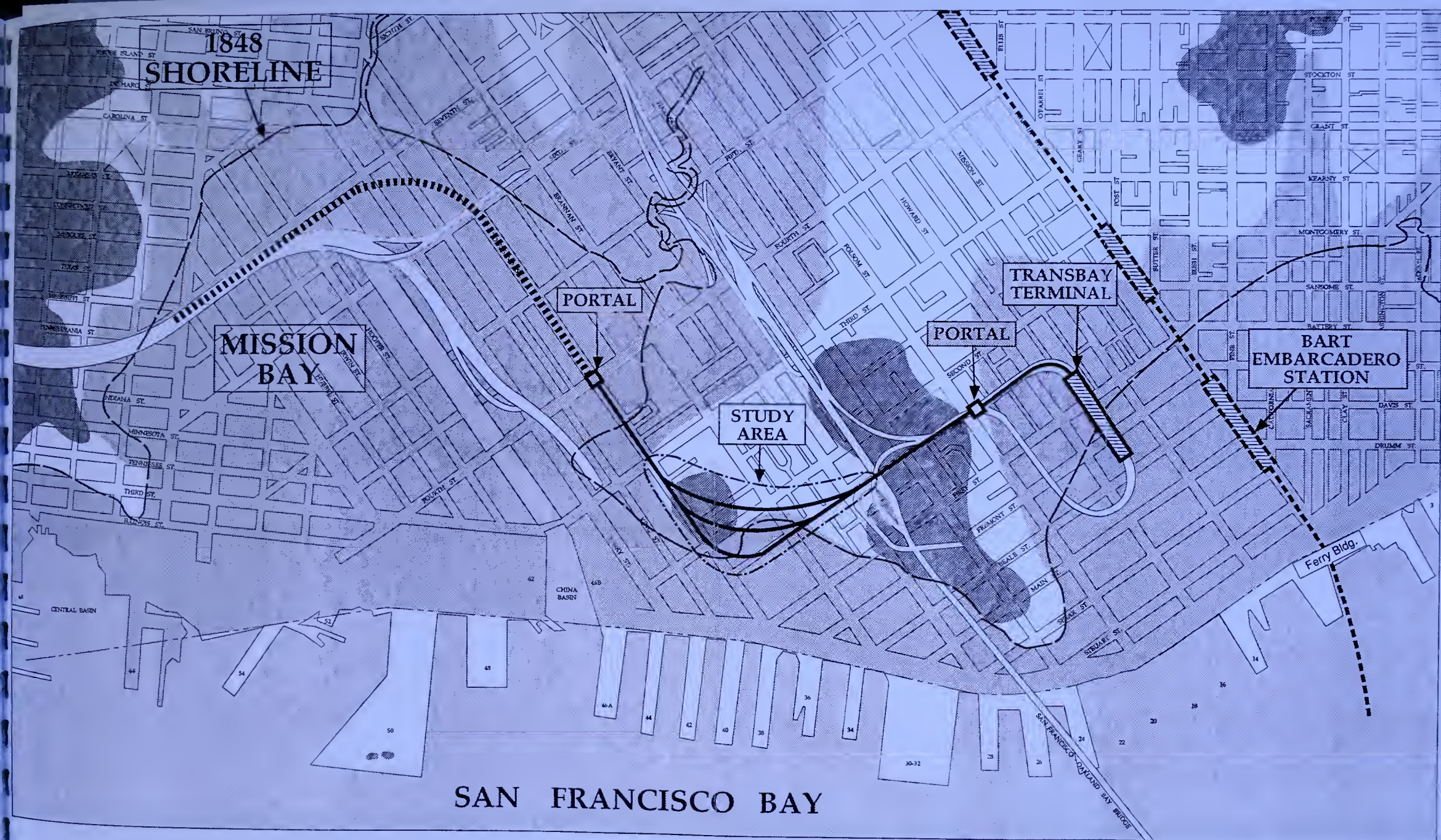
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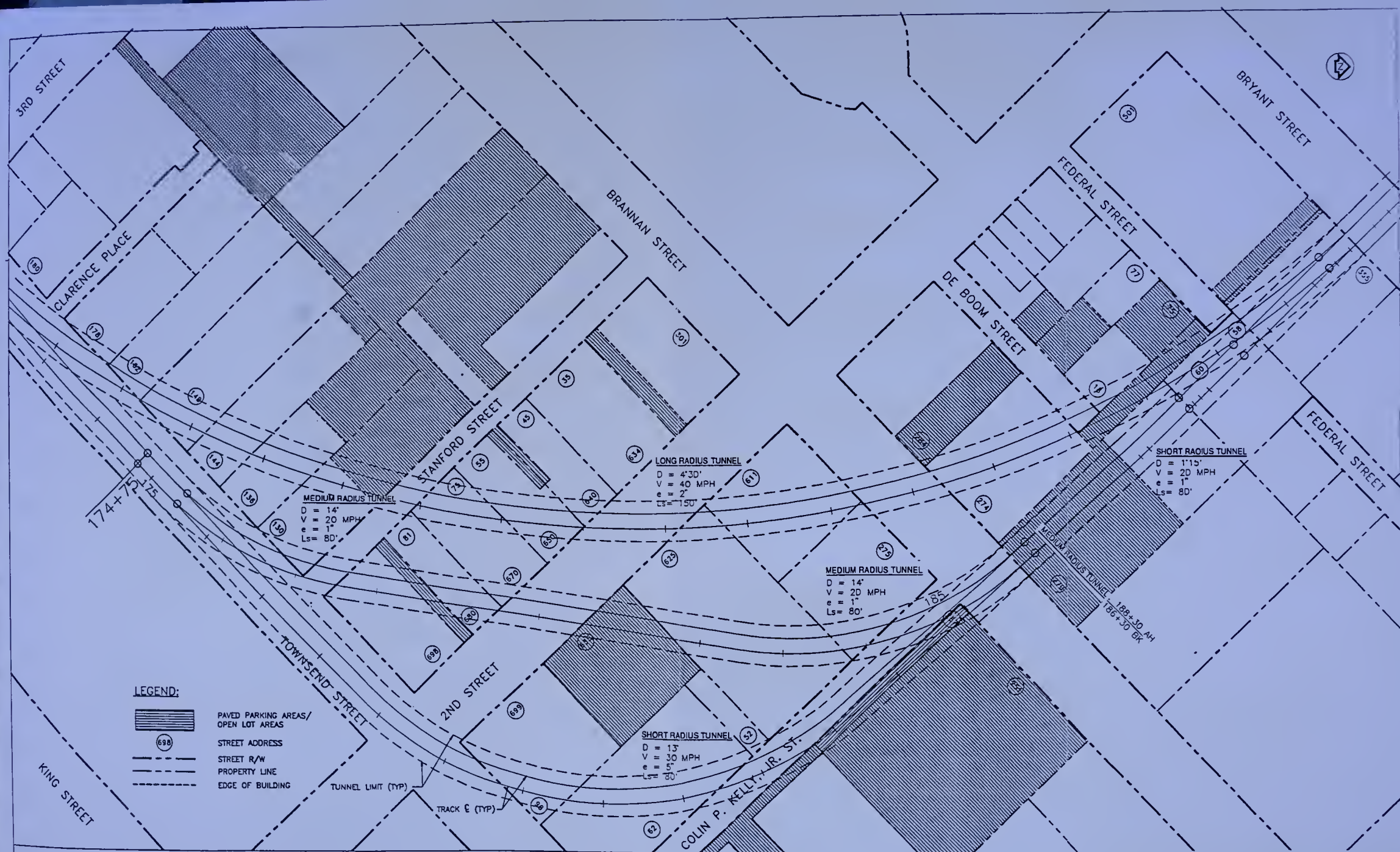


0 600  
Scale in Feet

**KEY**

	Fill
	Rock Outcrop





ICF KAISER ENGINEERS, INC. — DE LEUW CATHER & COMPANY  
SAN FRANCISCO, CA

CALTRAIN SAN FRANCISCO DOWNTOWN STATION RELOCATION  
EIS/EIR PROJECT



PENINSULA CORRIDOR JOINT POWERS BOARD

TUNNEL ALIGNMENT ALTERNATIVES BETWEEN  
3RD STREET & COLIN P. KELLEY JR. STREET

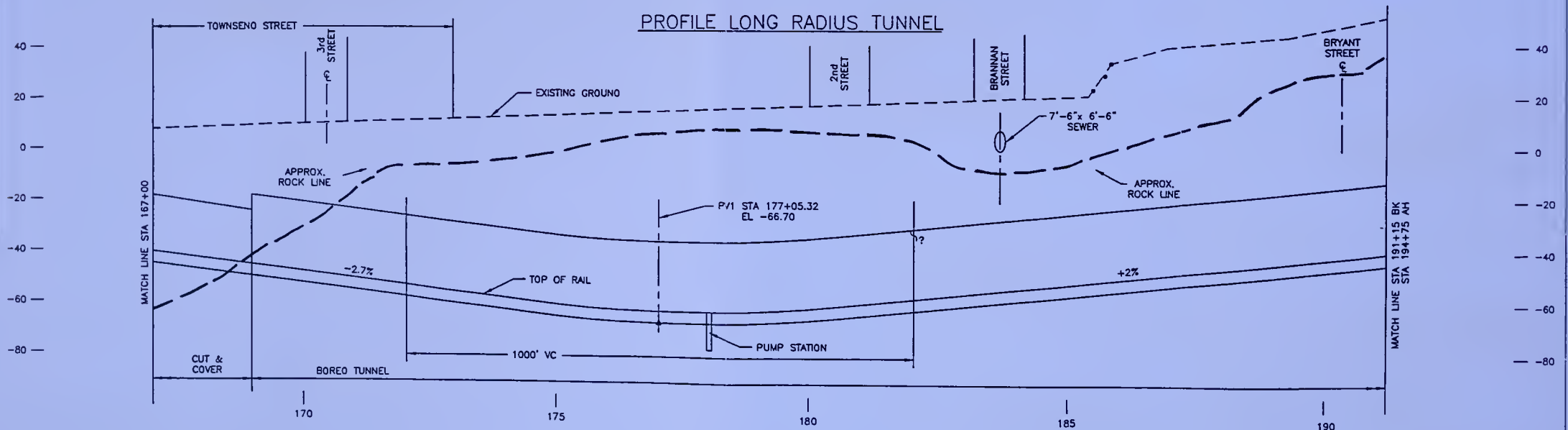
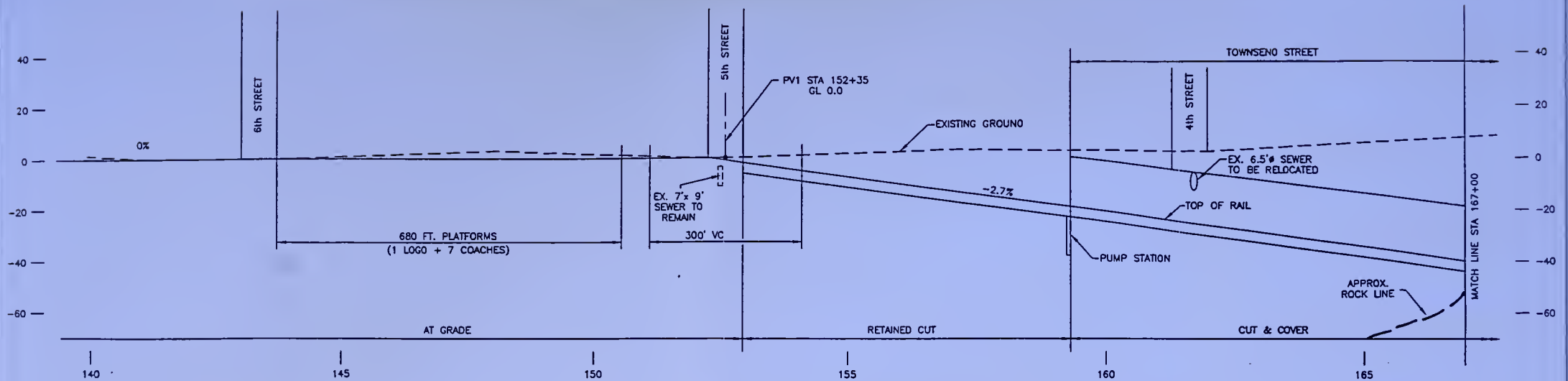
ICF Kaiser/De Leuw, Cather & Company  
Caltrain S.F. Downtown Extension  
San Francisco, California

DAMES & MOORE

PLATE 2



# PROFILE MATCH SECTION LONG, MEDIUM, & SHORT RADIUS TUNNELS



0 100 200 300 400  
HORIZONTAL SCALE IN FEET

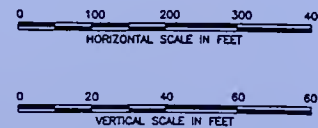
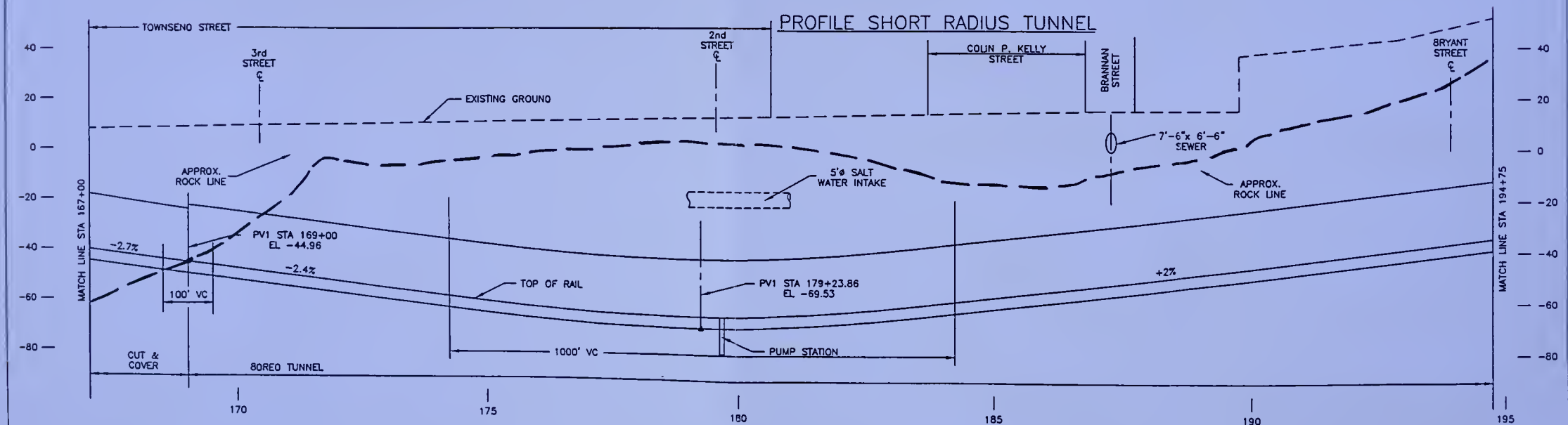
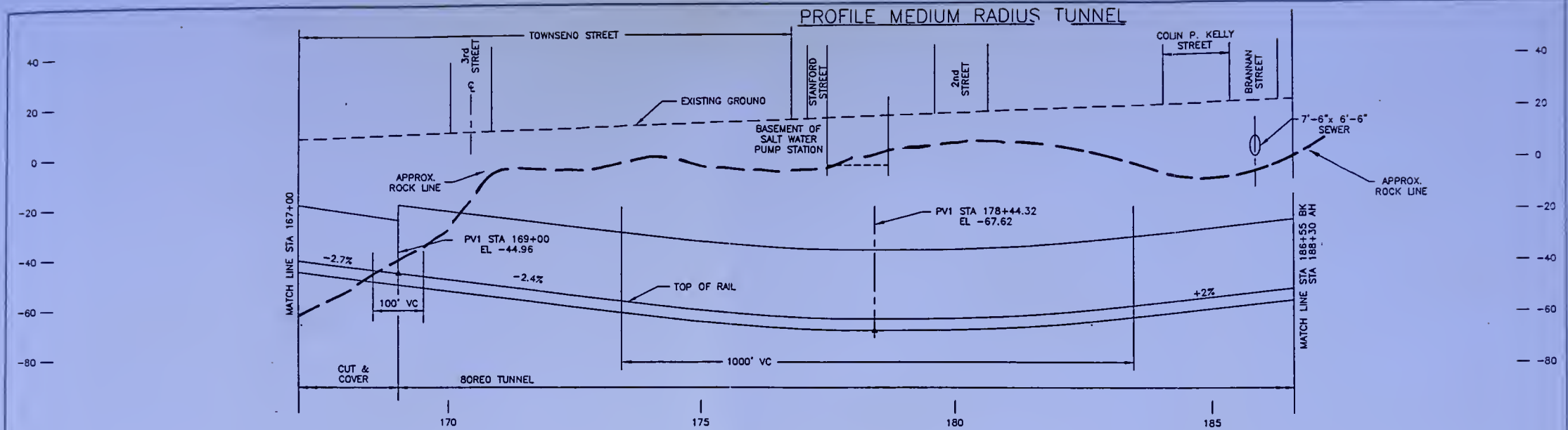
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VERTICAL SCALE IN FEET

## TUNNEL PROFILES AND ROCK LINE: "LONG RADIUS" TUNNEL AND CUT-AND-COVER EXTENSION ALONG TOWNSEND STREET

May 1996  
30751-005-003

ICF Kaiser/De Leuw  
Caltrain S.F. Downtown Extension  
San Francisco, California

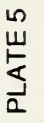




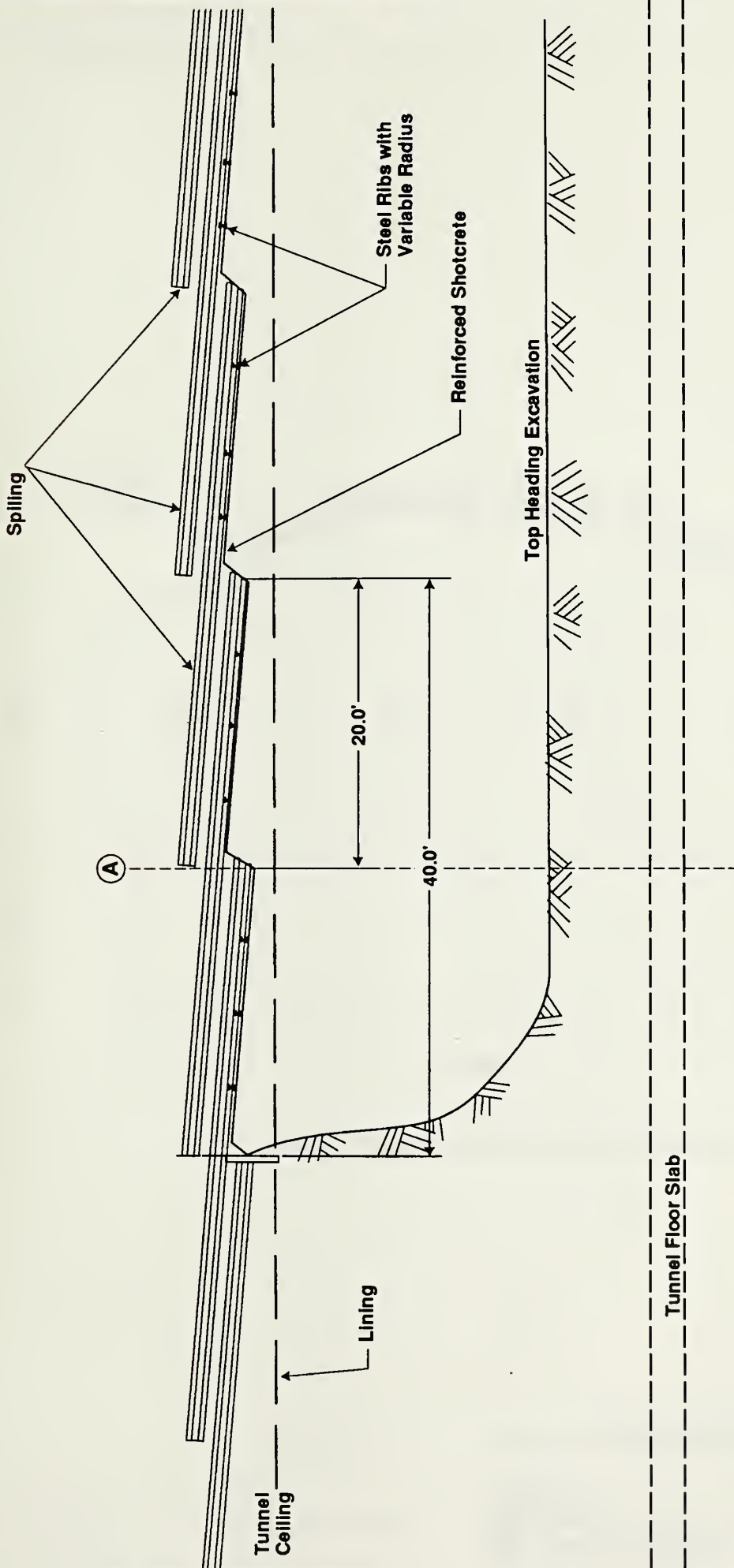
**TUNNEL PROFILES AND ROCK LINE:  
"SHORT RADIUS" AND "MEDIUM RADIUS"  
TUNNEL ALTERNATIVES**

ICF Kaiser/De Leuw  
Caltrain S.F. Downtown Extension  
San Francisco, California  
May 1996  
30751-005-003







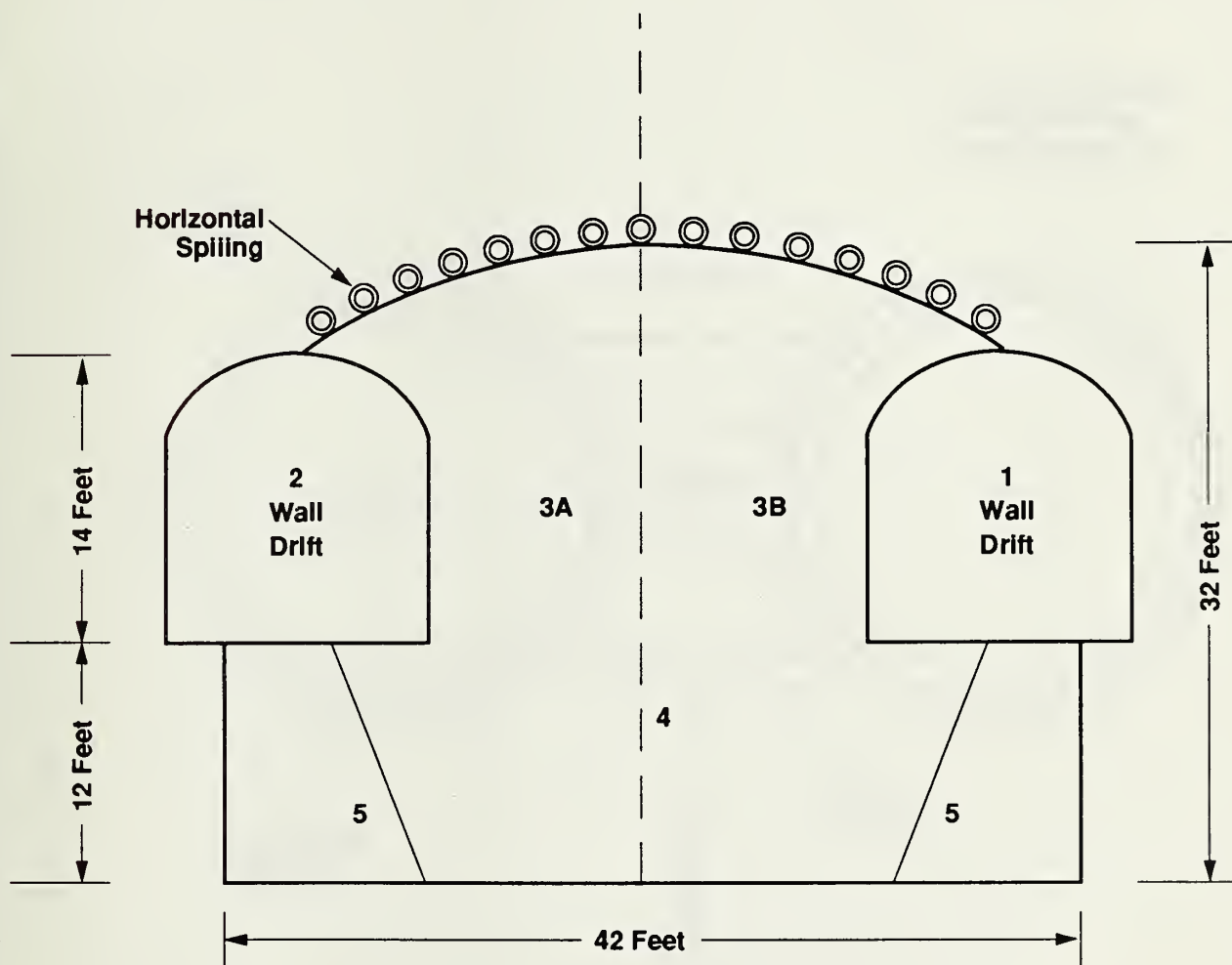


**GROUND REINFORCEMENT:  
LONGITUDINAL SECTION WITH  
PROPOSED SUBHORIZONTAL SPILING**

May 1996  
30751-005-003

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San Francisco, California





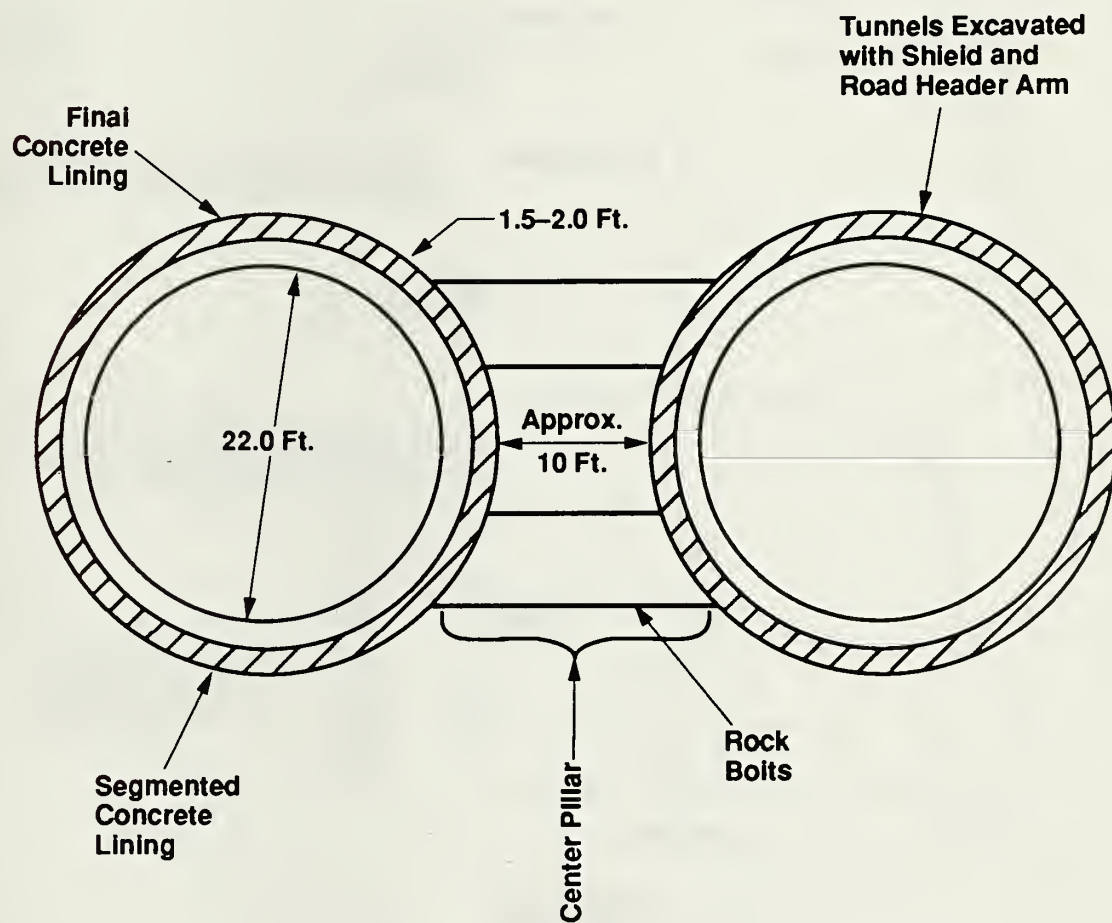
Note: Numbers Represent the Sequence of Excavation

# SINGLE TUNNEL WITH TWO WALL DRIFTS

May 1996  
30751-005-003

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Caltrain S.F. Downtown Extension  
San Francisco, California





#### **TWIN BORE, SHIELD-DRIVEN TUNNEL ALTERNATIVE**

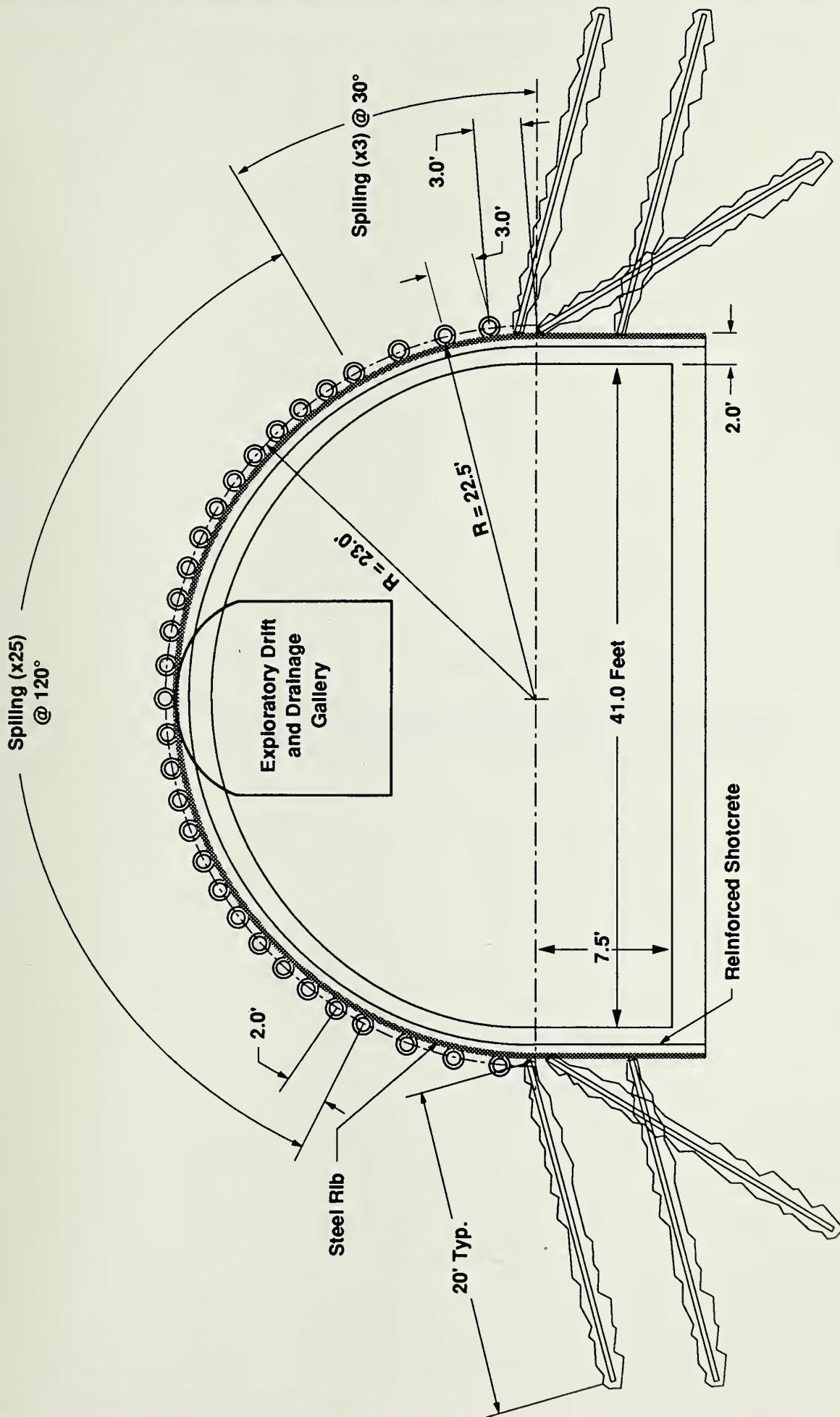
May 1996  
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ICF Kaiser/De Leuw  
Caltrain S.F. Downtown Extension  
San Francisco, California









# RECOMMENDED TUNNELING OPTION

May 1996  
 30751-005-003

ICF Kaiser/De Leuw  
 Caltrain S.F. Downtown Extension  
 San Francisco, California



**APPENDIX A**

**LETTER FROM PROFESSOR THOMAS D. O'ROURKE:  
REVIEW COMMENTS ON TUNNELING METHODS**



10 Twin Glens Road, Ithaca, New York 14850  
607-272-4029

EXPRESS MAIL

23 May 1996

Dr. Demetrious Koutsoftas  
Dames & Moore  
221 Main Street, Suite 600  
San Francisco, CA 94105

RE: CalTrain Commuter Rail Downtown San Francisco Extension: Tunneling Methods

Dear Demetrious,

This letter report provides input about tunneling methods for the referenced project, and should be regarded as providing viewpoints that supplement my letter report of 26 March 1996. It has been prepared after reviewing the following reports: 1) April 1996 report prepared by the Nicholson Construction Company (NCC), entitled "Ground Treatment and Support Feasibility Study", 2) April 1996 report prepared by Haley & Aldrich (H&A), entitled "CalTrain Downtown Station Relocation", 3) May 1996 report prepared Werner and Associates, entitled "Underpinning of Existing Historic Buildings", and 4) May 15, 1996 draft report prepared by Dames & Moore, entitled "Tunnel Engineering and Cost Estimate CalTrain S.F. Downtown Extension". The preparation of this letter report has not involved the other two members of the Board of Consultants, Mr. Norman A. Nadel and Dr. Tor L. Brekke, and therefore reflects specifically my own observations.

Many of my observations about alternate tunneling methods and routes have been conveyed to you by phone. This letter is intended to address briefly several important items associated with tunneling methods, mixed face conditions, and building protection.

### **Tunneling Methods**

The single mined tunnel options proposed by NCC and H&A are technically feasible, and with good workmanship and engineering supervision, are appropriate for stability and ground movement control. Each method involves carefully staged excavation and support, and therefore can be sufficiently flexible to accommodate the variable ground conditions that will be encountered during tunneling. Whereas shield driven tunneling represents an interesting option that has been employed recently in San Francisco (Richmond Transport Tunnel), its use on the CalTrain Project involves uncertainties associated with adaptation to variable ground, narrow pillar between tunnels, separate drives, mobility and maneuverability of the shields, and loss of flexibility in responding to changes in ground. Given the very small difference in estimated cost between the single mined and twin shield tunnel options, it is reasonable to focus planning and



continuing design on a single mined tunnel. The method that provides for systematic spiling installed ahead and around the tunnel face to form an "umbrella arch" has many attributes, especially when Scheme B proposed by NCC is considered for overlapping crown support in sensitive areas. When systematic spiling is combined with a central pilot drift, there is the opportunity for pre-drainage and pre-support, as well as advance exploration of potentially unfavorable soil and rock.

### **Mixed Face Conditions**

As stated in several of the reports that were reviewed in preparation for this letter, and as emphasized in Board of Consultants reports, the tunneling will be performed in mixed face conditions. This means that both "rock-like" and "soil-like" materials below the water table will be encountered. The potential for both soil and weathered, weak rock is especially troublesome because such conditions can promote rapid ground loss, driven by seepage gradients from groundwater discharging into the tunnel.

The current tunnel profiles have been established at a depth where the estimated minimum rock cover varies from approximately 10 to 20 ft. This estimation is based on limited borehole information. Zones of local, concentrated rock weathering may result in "soil-like" material extending below the proposed tunnel crown, and locations of ancient stream and drainage channels may be filled with saturated soils locally deeper than implied by the average or estimated minimum top of rock.

Assuming that the buildings are supported on shallow foundations directly bearing on rock (H&A report, p. 6) can lead to overly optimistic expectations. Care must be exercised in evaluating building response. It is likely that some foundations are supported partly on soil, weathered rock, and relatively sound rock. Because of the contrast in stiffness and bearing capacity associated with variable ground conditions, zones of concentrated building distortion are more likely to occur in response to tunneling-induced settlement and lateral displacement. The key to success in this case will be to control the movement of soil, particularly locally saturated soil zones, as well as to control movement of the rock mass. A key aspect, and perhaps the most critical aspect of mixed face tunneling, is control of soil and highly weathered rock movement, especially where perched or locally high regional groundwater exists.

The use of spiling, particularly overlapping, grouted spiling, is well-suited for mixed face conditions because it provides relatively continuous support of the crown well in advance of the face. Because access to suitable locations for surface dewatering will be restricted, it is very important to provide for drainage from the tunnel. The use of a pilot bore will aid and abet this process.



**Building Protection**


At this stage of the project, it is advisable to consider underpinning as part of a strategy for offsetting the effects of tunneling on existing structures. Accordingly, it is appropriate to provide for the expense of underpinning in the cost estimates of the project. The information provided in the Werner and Associates report appears to provide a reasonable basis for these estimates. Given the anticipated difficulties in site access for surface dewatering, it is likely that there will be access difficulties for installation of both control and drilled pin pile underpinning. It may be advisable, therefore, to provide for extra contingencies in the estimated underpinning costs, especially in the categories of control pier or column pick-up installation, monitoring, and mini-pile installation.

The Werner and Associates report states that no matter how a building is underpinned, there cannot be a guarantee that the building would be unaffected by a cave-in of the tunnel heading. The risk of building damage obviously increases as the number of buildings crossed by the tunnel increases. It is appropriate in this case to view risk as being compounded at an increasing rate relative to the number of buildings crossed. Given that the building at 625 2nd Street is only nominally affected by the short radius tunnel, the number of buildings crossed more than doubles and then quadruples as the medium and long radius tunnels, respectively, are considered relative to the short radius option.

It is because of the significant decreases in risk that the Board of Consultants has favored strategies that reduce the exposure of surface structures to the potential effects of tunneling. One of the most important decisions for the project will be selecting the tunneling route that balances appropriately the risks of disturbing existing buildings with the operational and cost advantages of the candidate alignments.

Should you have any questions, or seek further clarification of the issues covered in this letter report, please don't hesitate to contact me.

Sincerely,

A handwritten signature in black ink that reads "Tom O'Rourke". The signature is fluid and cursive, with a long horizontal stroke extending from the end of the name.

T.D. O'Rourke

TDO/m



**APPENDIX B**  
**REPORT BY HALEY & ALDRICH**  
**ON TUNNELING METHODS**



**FINAL REPORT ON  
CALTRAIN DOWNTOWN STATION RELOCATION**

by

**Haley & Aldrich, Inc.  
Denver, Colorado**

for

**Dames & Moore  
San Francisco, California**

**File No. 20249-000  
April 1996**





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7	Twin Bore Shield Driven Alternative
8	Twin Bore Conventional Excavation Alternative



## I. INTRODUCTION

This report provides Haley & Aldrich's input with regards to tunneling under existing historic buildings for alignment 3 AB-T between Colin P. Kelly and Townsend Streets for the Caltrain project as shown on Figure 1. This alignment curves under buildings which were built in the period between 1870 to 1900 and which are considered historic by the Historical Society. It is desired to go underneath these buildings without causing any damage to the overlying structures and Haley & Aldrich was contracted to assist Dames and Moore in assessing tunneling techniques to be employed such that damage to the overlying buildings would be minimal. The scope of work performed by Haley & Aldrich for this report is as follows.

- Task 1            Review available geological information from previously prepared reports with emphasis on rock quality and groundwater conditions.
- Task 2            Evaluate potential tunneling methods which have been proposed and which could be used to accomplish the desired opening.
- Task 3            Discuss remedial measures that may be necessary to insure the integrity of overlying buildings during the tunneling operation including ground improvement techniques such as grouting, dewatering, or rock reinforcement.
- Task 4            Prepare a brief letter report summarizing the above work items.

This report will be used as part of a larger EIR/EIS document for the Caltrain project, the specific emphasis of which will be in the area of historic buildings located between Colin P. Kelly and Townsend Streets. Haley & Aldrich was tasked to look at three different possible alignments in this area to assess which alignment would best suit the project and have minimal impact on the buildings above. These alignments are referred to as the long, medium and short radius tunnel alignments as shown on Figures 2 and 3. Additional work is being performed with regard to ground improvement by Nicholson Construction Company and with regard to underpinning by Werner and Associates.



## II. BACKGROUND INFORMATION

Dames and Moore has performed geologic investigations along the proposed tunnel alignments which are documented in a geotechnical report entitled "Final Report Geotechnical Site Investigation, CALTRAIN S.F. Downtown Station Relocation EIS/EIR", dated September 25, 1995. Additionally, a Board of Consultants was convened to address technical issues associated with the project and the Board recommended that a single tunnel opening with sequential face construction be performed. Although the method recommended by the Board is conservative and feasible, the Board acknowledged that alternative excavating procedures would be equally valid. In addition, Haley & Aldrich was asked to make an independent evaluation of the possibility of using two separate tunneled openings.



### III. SUMMARY OF SUBSURFACE CONDITIONS

#### 3.1 Site Geology

Geologic conditions in the vicinity of the tunnel alternatives were ascertained from test borings drilled in the area in June, 1995 (borings B-2, B-3, B-8, B-9, and B-10). Geologic units encountered in the borings were primarily unconsolidated materials (fill or clay), shale, and sandstone. The thickness of unconsolidated material was greatest in borings B-3 (30 ft.) and B-8 (22 ft.) located along Brannan and Bryant Streets, respectively, between First and Second Streets. The thickness of unconsolidated materials in the other borings was only a few feet. The shale encountered in the borings was primarily black, weathered, and highly fractured or sheared. The sandstone units were also highly fractured, but some zones of relatively intact rock were also encountered. Ground water levels are typically 10 ft below existing ground level.

Based upon the results of these test borings, it appears that all three of the proposed long, medium, and short radius tunnel alternatives below historic structures will be located in highly fractured rock with little unconsolidated cover.

#### 3.2 Rock Mass Classification for Tunneling

Ground behavior during rock tunneling has been described in the engineering literature with a variety of classification systems. These systems utilize quantifiable rock mass properties to characterize rock masses for tunneling purposes. Given below is a summary of each geologic unit utilizing three different classification systems. There is a considerable degree of variation and uncertainty in the use of these systems and, therefore, a high degree of care is warranted in their use. Additionally, the classification systems are based on the evaluation of a combination of factors, although rock mass behavior is often dominated by just one factor. Therefore, the use of these classifications must be accompanied with a thorough understanding of the controlling characteristic of the rock mass, rather than simply the overall classification. Finally, rock mass classification systems are based primarily on drill-and-blast excavation methods, although the interpretations have been modified in some cases to include TBM excavation.

The classification systems utilized include the following:

- **Tunneling Categories:** Three broad categories of ground relative to tunneling were presented by Deere, Merrit, and Cording. The categories of "Good", "Average to Difficult" and "Very Difficult to Hazardous" are based on general descriptions of ground conditions and on other classification systems. The authors presented general behavioral characteristics for each category and also excavation and support considerations.
- **Geomechanics System:** The geomechanics system was developed by Bieniawski and is based on the sum of six numbers derived from various rock mass characteristics: uniaxial compressive strength, RQD, spacing of discontinuities, condition of discontinuities, groundwater conditions, and orientation of discontinuities. The resulting Rock Mass Rating (RMR) number can vary from 0 to 100, is used to give a



general classification for the rock mass, and can be used to estimate stand-up time and support requirements.

- **The Q-System:** This was developed by Barton, Lien, and Lunde and is based on the product and quotient of six numbers derived from various rock mass characteristics: RQD, number of joint sets, joint roughness, joint alteration, joint water conditions, and stress factor. The resulting number can vary from .001 to 1,000, provides a general classification for the rock mass, and can be used to estimate support requirements.

Sandstone comprises approximately 50 percent of the bedrock sampled by the four core borings drilled in the vicinity of the proposed tunnel alignments. Ground conditions within the sandstone were categorized as "Very Difficult to Hazardous" for 72 percent of the core samples. One zone of relatively intact sandstone was encountered, which resulted in 28 percent of the core samples being categorized as "Good". Using the Geomechanics System for the typical "Very Difficult to Hazardous" conditions, a RMR of approximately 32 was calculated, indicating "Poor" quality rock. Using the Q-System, a rating of approximately 0.02 was calculated, indicating an "Extremely Poor" rock mass.

Shale also comprised approximately 50 percent of the bedrock sampled by the four core borings drilled in the vicinity of the proposed tunnel alignments. Ground conditions within the shale are categorized as 12 percent "Average to Difficult" and 88 percent "Very Difficult to Hazardous". Most of the core samples (85 percent) had an RQD of zero. Using the Geomechanics System for the typical "Very Difficult to Hazardous" conditions, a RMR of approximately 25 was calculated, indicating "Poor" quality rock. Using the Q-System, a rating of approximately 0.02 was calculated indicating an "Extremely Poor" quality rock mass.

### 3.3 Groundwater Conditions

As has been pointed out by the Board of Consultants and others, the groundwater condition can have a significant impact on tunneling, and serious consideration must be given to groundwater control during design. In general, groundwater affects tunneling in three important ways:

1. Water inflow can interfere with tunneling and add both to the cost and the time for construction. Large, unanticipated inflows can be especially disruptive to the tunneling operation.
2. Water pressure in the rock mass simultaneously adds to the load imposed on support elements and reduces the rock's ability to help resist those loads by reducing the frictional resistance of the rock. Hence, water pressure in the cracks and fissures of a rock mass makes the rock much more difficult to "control."
3. Water table drawdown caused by tunneling; either as a result of the tunneling operation itself or as a result of a program of preconstruction dewatering, can extend well beyond the zone of influence of tunneling and have adverse impacts on adjacent properties. Two common examples of such impacts are the consolidation settlement of soft soil and the rotting of wood piles upon exposure to air. Such impacts are possible at the subject site.



Although it is known that the proposed tunnels are located below the groundwater table, very little else is known about the magnitude or distribution of rock mass permeability at the subject site. Low RQD values, per se, do not necessarily equate to high permeability because the rock fissures and cracks (especially for shale) can be filled with clay. Conversely, if the permeability is low, then it will be that much more difficult and require that much more time to implement an effective program of construction dewatering.

The safest and most conservative design assumption to make at this time is that construction dewatering will be used and that the proposed tunnels will be constructed "in the dry." Some combination of deep wells, well points installed from the ground surface, or horizontal or vertical wells installed from inside the excavation could be used for this purpose. Also, tunneling techniques that either minimize the possible adverse consequences of the groundwater and/or facilitate the staged control of the water are preferable and should be utilized. Additional discussion about this topic and about other topics as they relate to tunneling are given later in this report.



#### IV. PROJECT LAYOUT

As discussed above and as shown on Figure 1, the proposed tunnel alignments for the project curve generally from Townsend St. to either Colin P. Kelly St. or Brannan St. depending on the radius of curvature chosen for the tunnel. The shortest radius tunnel would go beneath the fewest number of historic structures, but might introduce slower operating speeds on the project. The longest radius of curvature tunnel would go below the largest number of structures but would also be the most favorable from the point of view of operating speed. Proposed profiles for these three options are shown in Figures 2 and 3, and it is our understanding that these profiles are about as deep as possible and provide the greatest amount of rock cover (approximately 20 to 50 ft.) between the tunnel crown and the bottom of existing building foundations. It is assumed for the purpose of this report that the buildings are supported on shallow foundations bearing directly on rock. If this is the case, then the key to success for controlling building settlement will be to control the movement of and deformations within the rock mass itself.

The most basic requirement for this underground opening is to provide sufficient space for two parallel trains moving into and out of downtown San Francisco. Either one fairly large double-track opening, or two, smaller, single-track openings could be used for this purpose. Shown in Figure 4 are the excavated dimensions of a single opening being approximately 45 ft. wide and 33 ft. high at the crown. Also shown in Figure 4 is the construction sequence recommended by the Board of Consultants for this opening. As described by the Board, this opening would be built using one center drift and two parallel wallplate drifts followed by the sequential excavation and support of the full top heading and bench portions of the opening. Shown in Figure 5 is a second proposed construction sequence for this same opening which will be discussed later in this report in greater detail.

Figure 6 shows a third proposed construction sequence for this opening as detailed by Nicholson in their report entitled, "Ground Treatment and Support Feasibility Study". Given in Figures 7 and 8 are typical dimensions for twin tunnels excavated either mechanically or by drilling and blasting, respectively. Although the Board has pointed out that excavation of a rock tunnel by TBM at the subject site is probably not advisable, it is believed that an approximately 26-ft.-diameter (excavated), shield-driven tunnel with roadheader excavation and concrete segment support as shown in Figure 7 is possible and deserves at least conceptual level design consideration. If mechanical excavation is shown to be not feasible for these tunnels for some reason, then two drilled and blasted tunnels could be built as shown in Figure 8. In general, each of the drilled and blasted tunnels would have finished inside dimensions of approximately 22 ft. wide and 24 ft. high at the crown as shown in the figure.

Both the one-tunnel and two-tunnel options have advantages and disadvantages relative to use on the proposed project. The one tunnel option has the advantages that it can be excavated and supported sequentially, providing maximum flexibility to deal with ground conditions both as anticipated during design and as actually encountered during construction, and maximum opportunity to install presupport and prestabilization measures prior to expanding the opening to full width. This work is difficult, costly, and time consuming and requires the use of both very thoughtful design concepts and carefully controlled and high-quality construction operations. As has been pointed out by the Board, however, this single tunnel proposal is well within the realm of feasibility on the basis of currently available design and construction methodologies.



The two-tunnel option offers greater flexibility for laying out the project, since track alignments and profiles can be varied for each track, and provides greater opportunity to incorporate underpinning into the project if this should become necessary. The biggest disadvantages of a two-tunnel approach are the facts that a wider footprint for the work is needed because of the need for a central rock pillar (which itself must be carefully supported) and the need to traverse the alignment twice with two completely separate tunneling operations. Either approach is, however, feasible from an operational point of view and could be made to work at the subject site. It is believed that a two tunnel option is only feasible and should only be used if it can be shown that construction by shielded roadheader is possible. If it is necessary to use drilling and blasting to build the two tunnel option, then the single tunnel option offers many advantages as compared to two openings. Hence, a layout involving two drilled and blasted tunnels as shown in Figure 8 will not be considered in this report.



## V. TUNNELING TECHNOLOGY

### 5.1 General

The key to successful tunneling is always an ability to build the project in a safe and stable manner, at a reasonable cost, and in the shortest possible time. If major third party impacts are a problem, as they are at this site, then the issue of ground stability becomes of paramount importance. Any tendency for this highly fractured rock mass to disintegrate under the action of tunneling must be avoided both at the face of excavation and along the completed opening. Given in the following sections of this chapter are comments and suggestions about how ground control can be accomplished for the subject project for both the single and twin tunnel options.

Tunneling is generally thought of as a "highly" risky endeavor primarily because no one can know with certainty what type of ground will be encountered during construction or exactly how that ground will respond to a particular tunneling method or technique. For projects constructed in rural areas, almost all of the risks of tunneling are "internal" to the process; resulting in increased cost or time for the work but with minimal impact to third parties. This is definitely not the case, however, for tunnels constructed in urban areas where problems associated with tunneling can have significant impacts on adjacent and overlying infrastructure improvements or buildings, as well as having other community or environmentally-related impacts that must be addressed. For many urban tunneling projects, it is this array of third party impacts that controls, to a large degree, how the tunnel is both designed and constructed.

In order to build a tunnel, the Contractor must excavate the ground, he must carefully control the ground at or near the face during the process of excavation, and he must erect a lining as the tunnel advances so as to create a safe and stable working environment for the construction operation. Upon the completion of excavation and support, the Contractor returns to the tunnel in order to construct a final lining that fulfills the structural and operational criteria of the tunnel owner. Most of the risks associated with tunneling, and most of the concern for an urban tunneling project, occur during the early stages of construction when the ground is being excavated, controlled, and supported.

At the subject site, the rock is of very poor quality with low cover, the work will take place below the water table, the proposed opening is fairly large, and the tunnel is overlain directly by historic structures. Hence, the ground for this project will need to be extensively investigated for the purpose of tunnel design and extensively improved and/or strengthened both prior to and during the construction operation. As was discussed earlier in this report, one form of ground improvement that can and should be utilized for this project is construction dewatering in order to eliminate, to the greatest possible extent, the adverse consequences of groundwater flow and pressure on the tunneling operation. Other possible ground improvement techniques that need to be evaluated are various forms of grouting and rock reinforcement, staged excavation, and the use of excavation methods that minimize disturbance to the rock mass. Finally, Contractor prequalification, the continued use of a Board of Consultants, rigorous supervision of the work, and a conservative approach to design must all be considered as ways of further reducing the risk of tunneling to acceptable levels for the subject project.



## 5.2 The Single Tunnel Option

Given in Figure 5 is a proposed construction sequence for a single, large tunnel option that is similar to but different in some ways from the proposal advanced by the Board of Consultants. As was indicated by the Board, different approaches to construction of this opening are possible, and the layout shown in Figure 5 will be discussed so as to demonstrate some possible variations. Please note that all of these discussions are conceptual and that considerable additional subsurface investigation and design activities are necessary before a final option is chosen.

Shown in Figure 5 are two wallplate drifts but no center drift. Sufficient dewatering would be required to draw the water table down to at least 2 ft. below the proposed wallplate drift invert prior to construction, and it is assumed that this could be accomplished with fairly large diameter, deep wells distributed along the proposed alignment. Each drift would then be excavated and supported full face with whatever techniques and methodologies are best suited to the observed ground conditions. In general, roadheader excavation with light blasting and rockbolt and shotcrete support would seem to be appropriate. However, steel ribs and shotcrete could also be used. Spiling in front of the face for each round of each adit as recommended by Nicholson might be specified as an additional precaution. Finally, these two openings would be excavated at a minimum 50 ft. face offset in order to further reduce the possibility of adverse ground deformations.

There are many advantages to the construction sequence discussed above. First, the invert elevation of the wall plate drift is fairly high requiring less groundwater drawdown. Although it is not known with certainty at this time if a deep well arrangement could be used for this purpose, such an assumption is acceptable for cost estimating purposes. If necessary, the drift dimensions could be reduced to further enhance stability and the first drift could be used to facilitate dewatering for the second drift. Depending on rock conditions, it might even be possible to excavate these adits with roadheaders to reduce rock disturbance.

Upon the completion of wall plate drift excavation, these drifts are then available to be used for extensive ground improvement and ground prestabilization activities prior to any additional work being done on the opening. For instance, an array of rock bolts and a sequence of grouting activities could be used to strengthen and stiffen both the remainder of the crown and the sidewalls of the bench excavation. As has been pointed out by others, grout per se might not increase the "strength" of the rock mass to a great degree, but a good combination of rock-bolting and grouting could greatly enhance the "stiffness" of the rock mass and greatly reduce the rock mass deformations that would take place during full width and full depth excavation of the opening. The wallplate drifts could also be used both to erect a continuous reinforced concrete wallplate footing that could be used for support of steel ribs and to further augment and improve the dewatering layout so that the water table could be further drawdown to well below the invert elevation of the proposed bench. For instance, it would be possible to drill a row of wellpoints straight down through the invert of the drift to below the invert of the bench for dewatering purposes.

Once all work from the wallplate drifts is complete, then the top heading can be opened and supported in two stages as shown in Figure 5. A variation for bench excavation might be to remove only the center portion of the bench, and then support rock immediately adjacent to the sidewalls with rock bolts and shotcrete in 5 to 10 ft. long panels. Again, depending on the results of the final subsurface investigations, it might be possible to remove all or most of



this rock with a roadheader, although some of the higher quality sandstone might need carefully controlled blasting during appropriate windows of opportunity and with vibrations controlled to well below threshold levels necessary to cause building damage. In all cases, however, all work associated with construction of this opening will be the object of careful control, detailed inspections, and many instrumented observations of vibrations, deformations, and support element loads and stresses.

Haley & Aldrich, Inc., has had the opportunity to read a report entitled "Ground Treatment and Support Feasibility Study" prepared for the subject project by Nicholson Construction Company. In general, this report is a good overview of the types of prestabilization procedures that can be used to safeguard the historic structures in question. As was pointed out in the Nicholson report, these procedures can be implemented from the ground surface, from the face of excavation, from adjacent openings and/or from specially constructed shafts either at the portals or at carefully selected locations along the alignment. The Nicholson report also discusses a type of construction that depends heavily on preinstalled spiling, micropiles, and fiberglass rock reinforcement to advance the opening. This approach has been used for similar projects in other locations, and could certainly be adapted to the subject site. Estimated construction costs were evaluated for Nicholson's Scheme B for a single opening as shown in Figure 6. For further information, the reader of this report is referred to the Nicholson document.

Although it is difficult to make projections at this early stage in the work, if the opening is made as discussed above and with many of the ground improvement techniques discussed by Nicholson, the vast majority of the alignment would experience a ground surface settlement of 1.0 in. or less. For a "worst possible" tunneling scenario, a settlement of no more than 2.0 in. should be possible. Inevitably, however, the question is always raised about what is the possibility of a catastrophic situation developing at the subject site wherein large settlements occur causing significant damage to the historic structures. Although it is imprudent to say that a catastrophe is not possible under any circumstances, the possibility of something catastrophic happening while using the procedures and techniques described above is highly unlikely.

As a fall-back position, detailed ground response observations with appropriate feedback to construction personnel and thorough contingency planning could be used to further reduce the risk of a complete collapse.

### **5.3 Twin Tunnels**

Shown in Figure 7 is a twin tunnel proposal that could be built utilizing a shield for installation of support and a roadheader for excavation. As the shield advances, spiling could be installed in front of the face, as necessary, both to control ground movements and to provide face stability. One technique that could be used would be to install spiling and perform grouting for a 30 to 40-ft. long reach of ground followed by excavation and support of say 25 ft. of tunnel. During production tunneling, it should be possible to obtain as much as 25 ft. of finished tunnel per heading for each three-shift working day, although the overall, average rate of tunnel advance would be less. Tunnel support would be provided from the rear of the shield, probably with expanded, precast concrete segments which would be back grouted against the rock during each shove.



One tunnel, recently constructed in San Francisco utilizing a similar approach, is the Richmond Transport Tunnel which was built in poor rock with a shielded TBM. It should be noted however, that the Richmond Tunnel had more rock cover than the proposed Caltrain Tunnel. The advantages of using a shield are that it provides immediate temporary support for the rock mass, a safe working area for the tunneling crew, and a solid working platform for drilling and grouting operations at the face. The precast concrete support rings provide a strong and safe tunnel lining during construction and a good, solid reaction for moving the shield forward. For the Caltrain Tunnel, it is proposed to use a roadheader for excavation rather than a TBM primarily because it is believed that the rock at the Caltrain site is of too poor a quality to support a TBM operation. Also, the use of a roadheader greatly reduces the cost of tunneling equipment, provides maximum access to the face for ground improvement activities, and still greatly minimizes disturbance to the rock mass as compared to drilling and blasting.

If it is decided that the drilling and blasting of this tunnel is necessary, then a horseshoe-shaped opening such as that shown in Figure 8 could be used. One advantage of the horseshoe-shaped opening is that has a greater compatibility with the shape of the train and results in a smaller width of opening and a smaller volume of excavation as compared to a circular opening. As with all of the tunneling schemes discussed in the report, however, some form of presupport and pregrouting of the ground will be needed in front of the face both to help stabilize ground at the face and to minimize movement of the rock mass. A top heading and bench method of excavation might also be needed to help control ground movement around the horseshoe tunnel because of the high sidewall of this opening.

Two items of extreme importance for both of the twin tunnel options discussed herein are the need to provide construction dewatering and the need to install an extensive system of presupport in the rock pillar between the two tunnels prior to excavation and support of the second tunnel. Since the shielded tunnel will be excavated full face, it will be necessary to provide the maximum, one-stage drawdown for this option. Drawing the water table down to below the invert elevation of the shield prior to the start of construction could be a problem for this site because of very limited access at the ground surface. Such is not the case for the horseshoe tunnel, since it could be built in stages, allowing the staged installation of the dewatering system as well.

With respect to the center pillar of rock, many case histories have shown that large surface settlements are possible if the pillar of ground between two parallel tunnels is allowed to deform during construction of the second tunnel. Hence the interior pillar must be strengthened and stiffened to accept the overlying ground load with a minimum of deformation.



## VI. CONCEPTUAL COSTS

Conceptual costs were developed for a twin shield driven tunnel option, a single tunnel option using two wall drifts, and a single tunnel option using longitudinal spiling as documented the Nicholson Report. These conceptual costs were for dewatering, mining and initial lining of the tunnel only. They do not include shaft construction or final lining. It is assumed that the final lining costs for the twin tunnels will be slightly higher since they have more area. The approximate costs per foot for a tunnel(s) 3,900 ft long are as follows:

Single Tunnel (Two drifts)	\$14,000/ft
Single Tunnel (Longitudinal spiling)	\$14,000/ft
Twin Tunnels (Shield driven)	\$13,000/ft

No contingency allowance is included in the costs above. The difference in cost between all options is less than 10 percent. Since many assumptions were made for each of these estimates, the accuracy is not within 10 percent. Therefore, costs of all the options are essentially the same. Production rates for all the tunnel options is expected to range between 10 to 20 ft per day. The cost estimates are so close that no real determination of a preferred alternative can be made at this time. Preliminary design should consider the three alternatives studied in this report. Further studies into the geologic conditions, project conditions and water conditions should be performed. Details of our cost estimates are also included in Appendix A of this report.



## VII. SUMMARY AND CONCLUSIONS

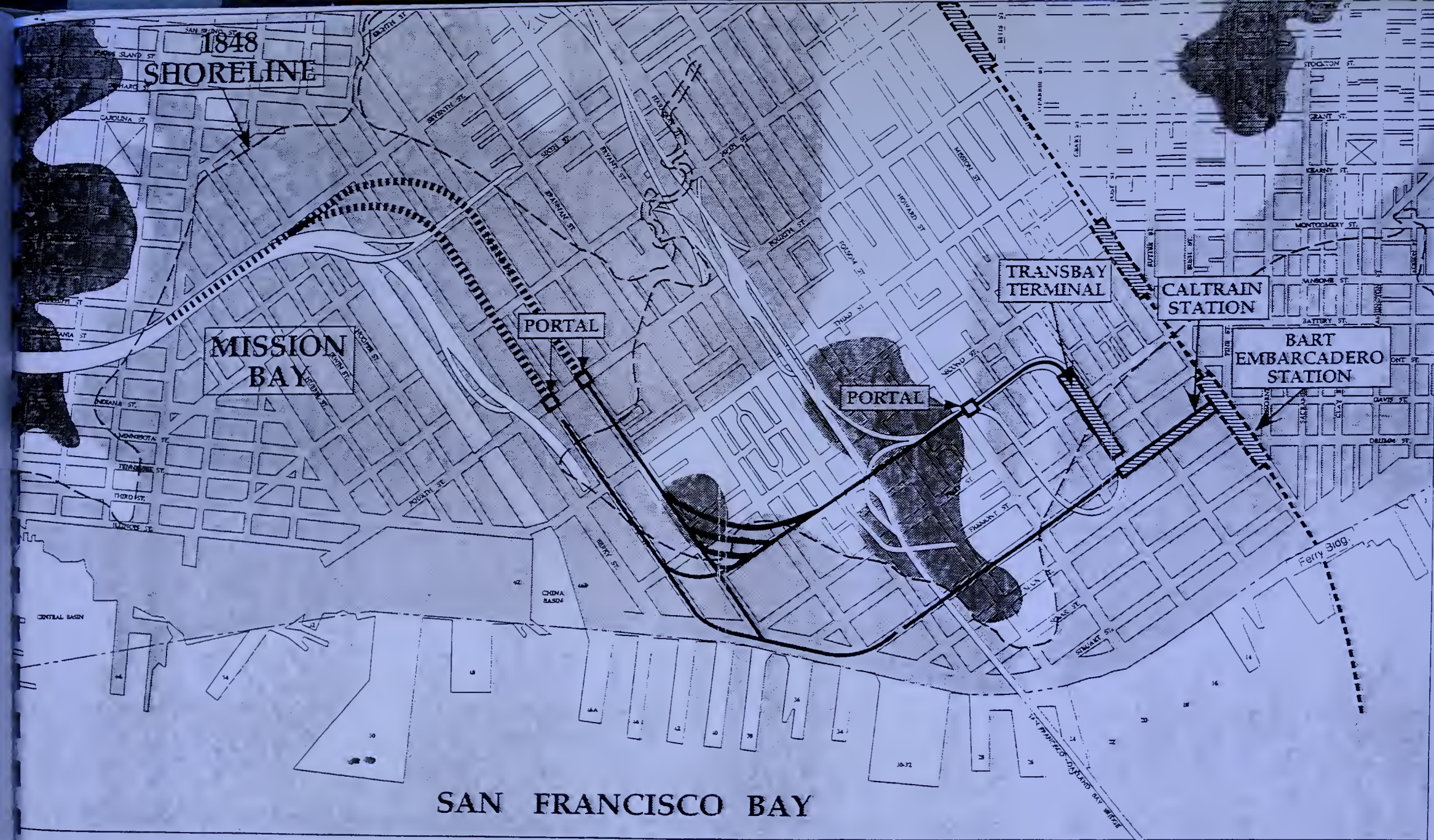
The primary purpose of this report is to discuss the important issues of concern relative to tunneling beneath historic structures. It is, however, difficult at this early stage to provide definitive recommendations about how to proceed with the work in the absence on site specific subsurface explorations. In general, either one large opening or two smaller openings could be used, depending on the quality of the rock mass and the applicability and assumed effectiveness of various ground improvement methodologies. One particularly important decision that must be made is how best to dewater the site prior to the start of construction. A second issue of concern is whether or not rock at the site can be excavated with a roadheader. Although some concern has been expressed about the ability of a roadheader to excavate the harder sandstone deposits, it is Haley & Aldrich's opinion that a roadheader can be used for this purpose even if additional cutter wear is experienced. As a minimum, it should be possible to keep the use of explosives in this highly sensitive area to an absolute minimum.

In conclusion, it is Haley & Aldrich's belief that tunneling beneath historic structures at the subject site is possible for the Caltrain project on the basis of currently acceptable design procedures and construction techniques. Either one large opening or two smaller openings could be used with the smaller openings being built in either a circular or a horseshoe shape, with or without the use of a roadheader. In all cases, construction dewatering and extensive ground pretreatment procedures will be required with particular emphasis on face stability and the amount of settlement that will occur above the tunnel. For the single opening, careful construction sequencing will be used for this purpose. For a two tunnel scenario, great care must be taken to stabilize and stiffen the central pillar of rock between the two tunnels. If done properly, it should be possible to limit settlements of the historic structures to 2.0 in. or less. The cost of all the options examined is within 10 percent. Based on the conceptual nature of cost estimates, all options are considered similar.



## **FIGURES**





LOCATION MAP

0 600  
Scale in Feet



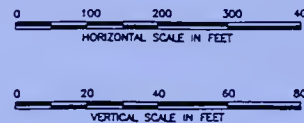
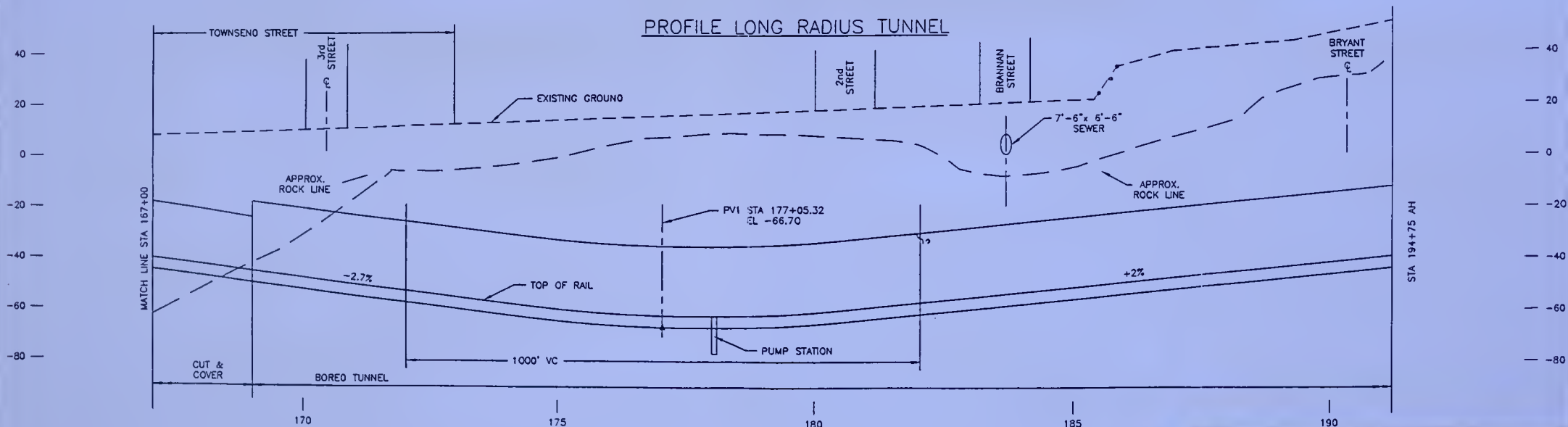
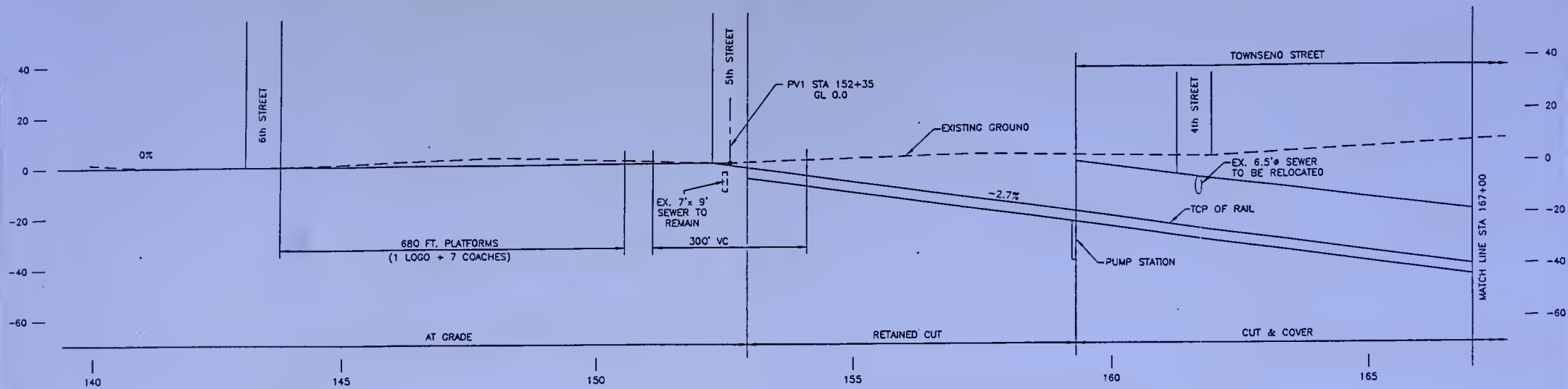
KEY	
	Fill
	Rock Outcrop

FIGURE 1



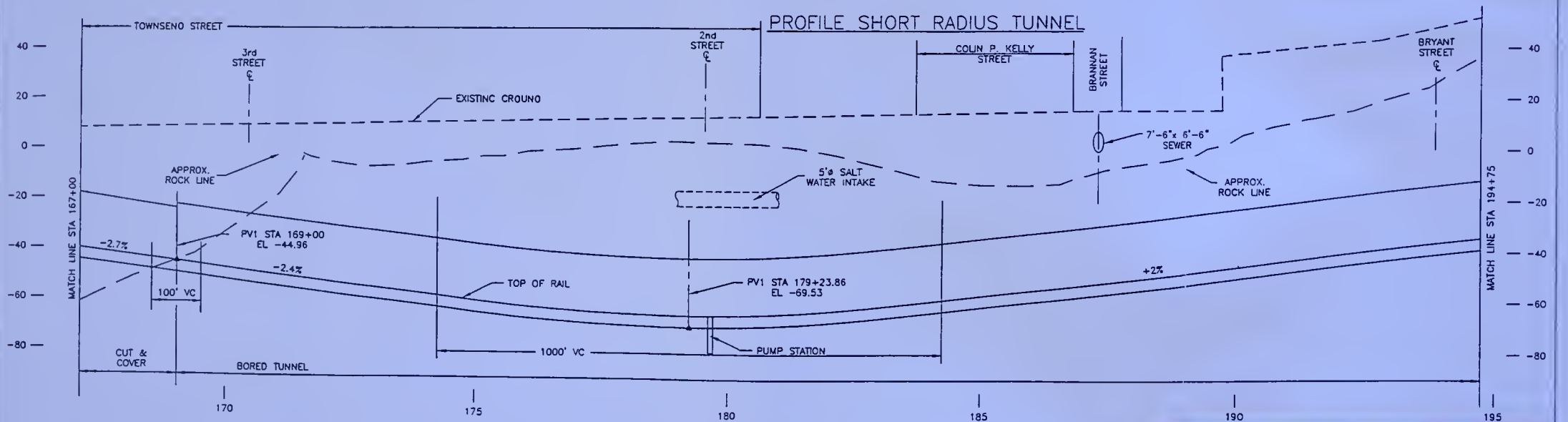
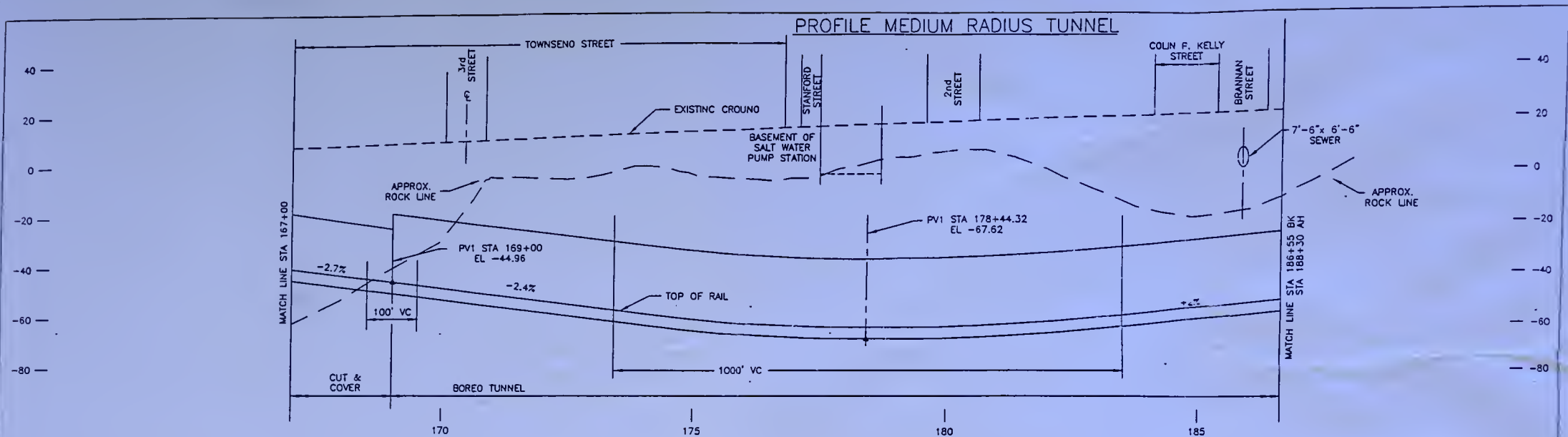
# PROFILE MATCH SECTION LONG, MEDIUM, & SHORT RADIUS TUNNELS



**HARRY & VIDRICH INC.**  
**Geotechnical Engineers & Environmental Consultants**  
 CALTRAIN EIR/EIS  
 TUNNEL PROFILE  
 PROJECT CITY AND STATE  
 MATCH SECTION LONG, MEDIUM, & SHORT  
 &  
 LONG TUNNEL  
 SCALE: AS SHOWN  
 MARCH 1996

FIGURE 2





0 100 200 300 400  
HORIZONTAL SCALE IN FEET

0 20 40 60 80  
VERTICAL SCALE IN FEET

**HALL & ALDRICH INC.**

**Geotechnical Engineers & Environmental Consultants**

CALTRAIN EIR/EIS  
TUNNEL PROFILE  
PROJECT CITY AND STATE

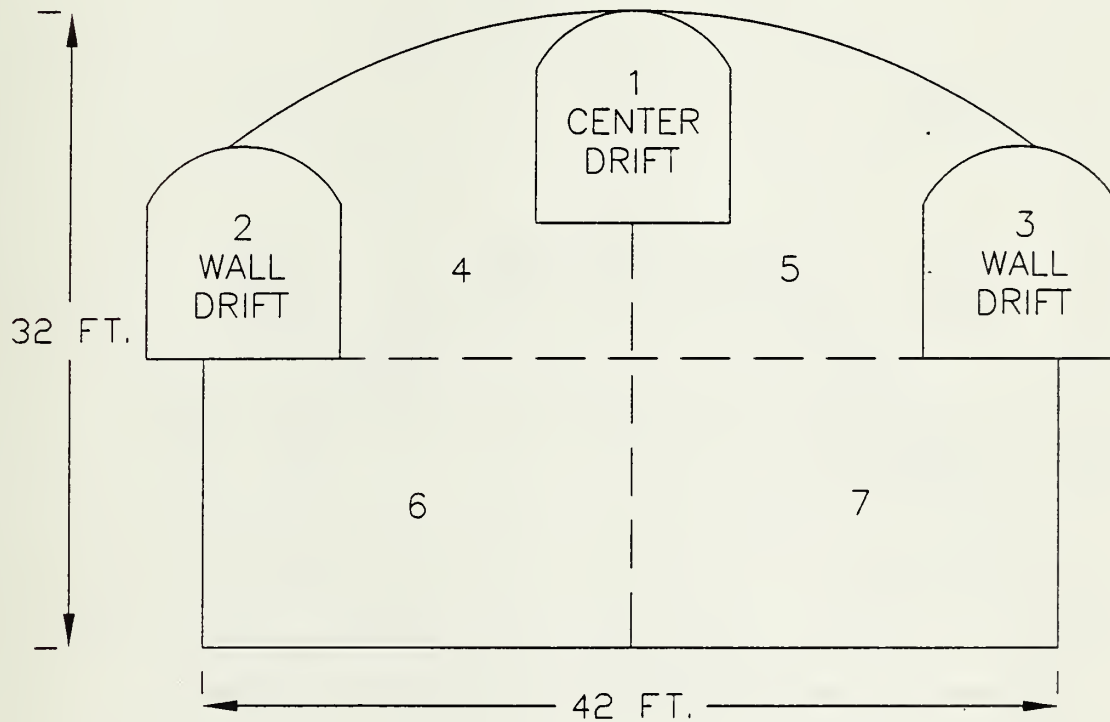
SHORT & MEDIUM TUNNELS

SCALE: AS SHOWN

MARCH 1996

FIGURE 3





NOTE: NUMBERS REPRESENT THE  
SEQUENCE OF EXCAVATION

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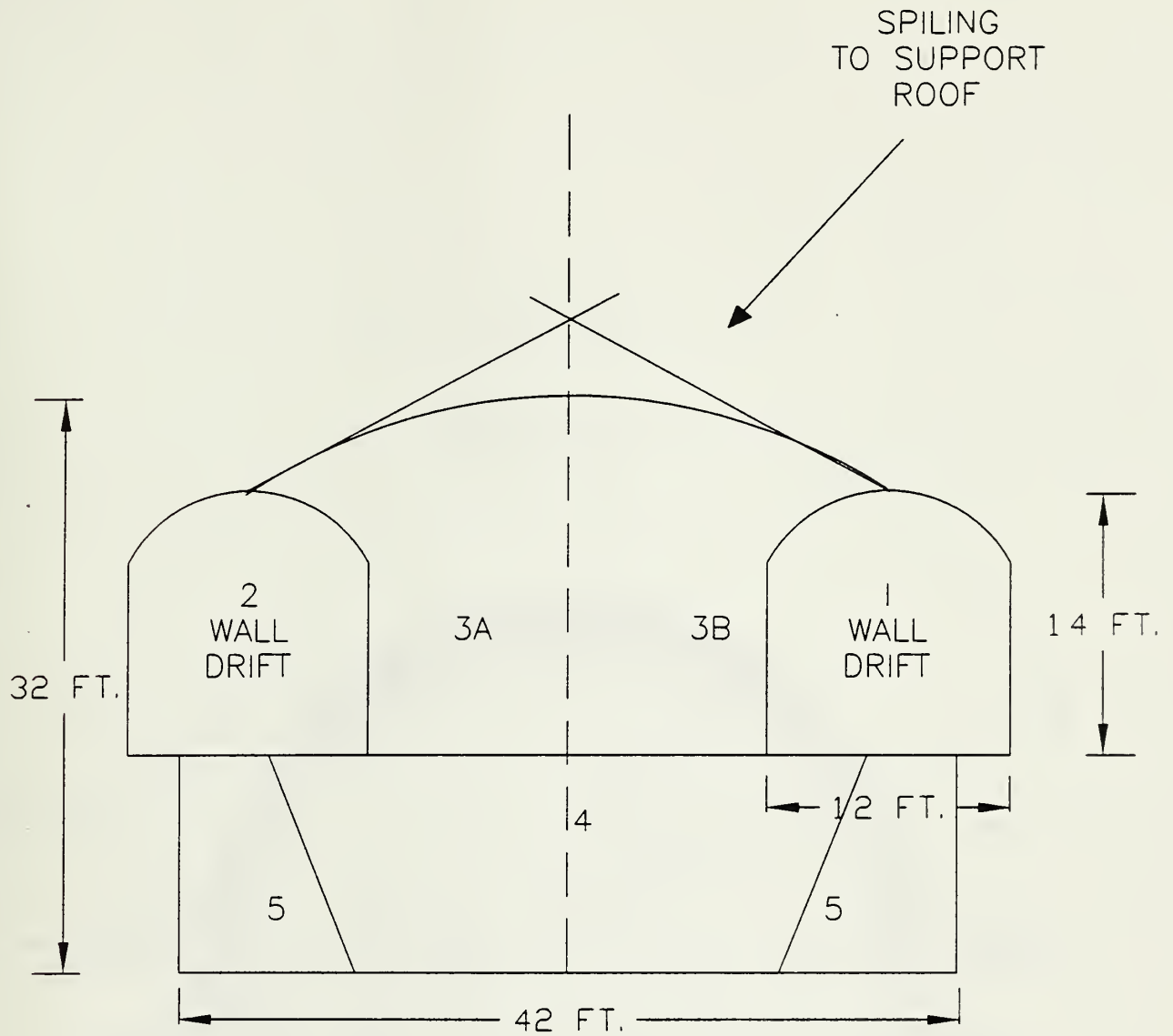
HALL & ADRICH INC	
	Geotechnical Engineers & Environmental Consultants
CALTRAIN EIR/EIS TUNNEL CROSS SECTION PROJECT CITY AND STATE	
TUNNEL BOARD OF CONSULTANTS ALTERNATIVE	
SCALE: NOT TO SCALE	MARCH 1996

FIGURE 4





NOTE: NUMBERS REPRESENT THE  
SEQUENCE OF EXCAVATION

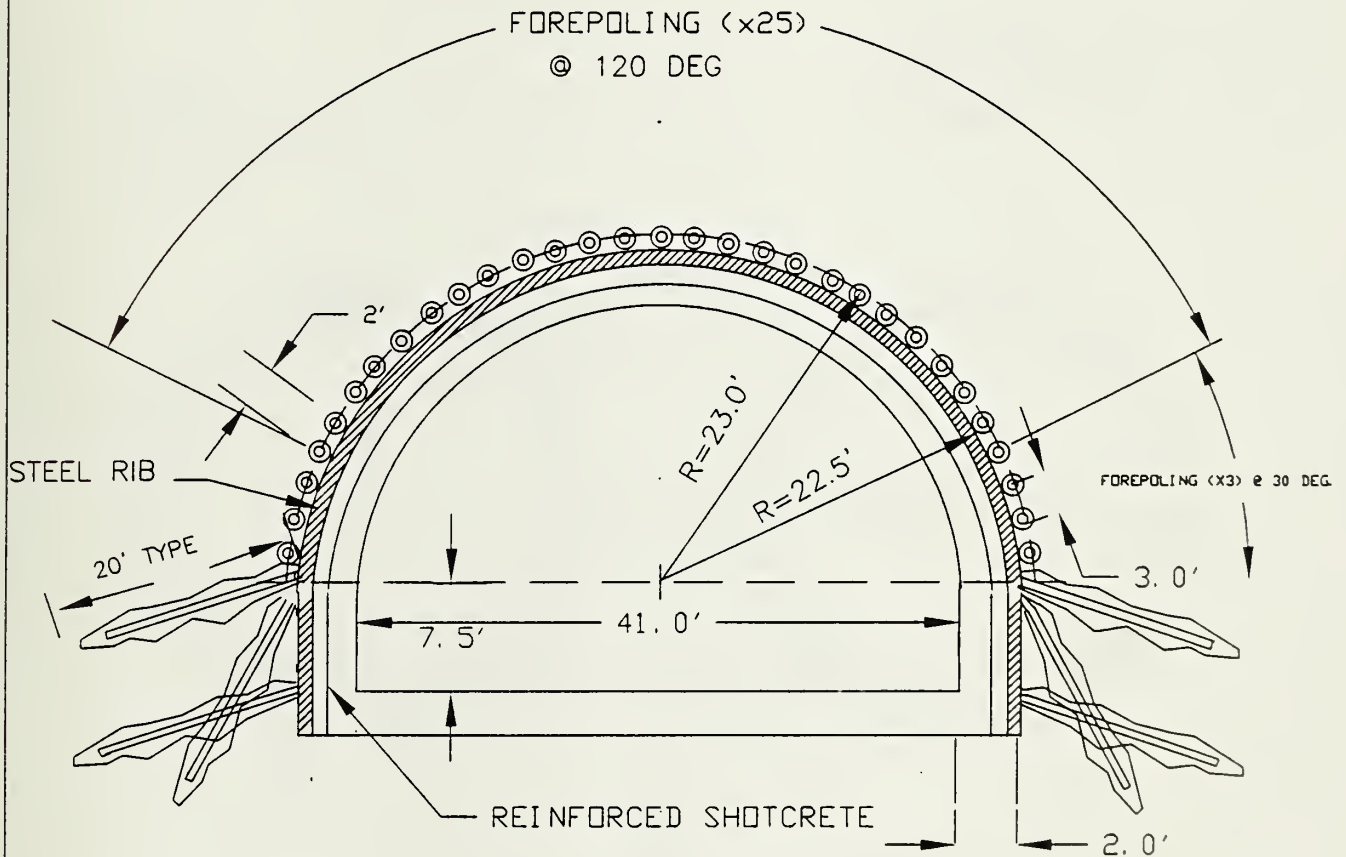
FILE NO. 20249-000 fig-4.dwg

HALEY & ALDRICH INC.	
<b>AGA</b>	Geotechnical Engineers & Environmental Consultants
CALTRAIN EIR/EIS TUNNEL CROSS SECTION PROJECT CITY AND STATE	
SINGLE TUNNEL ALTERNATE	
SCALE: NOT TO SCALE	APRIL 1996

FIGURE 5



TOTAL FOREPOLING = 31

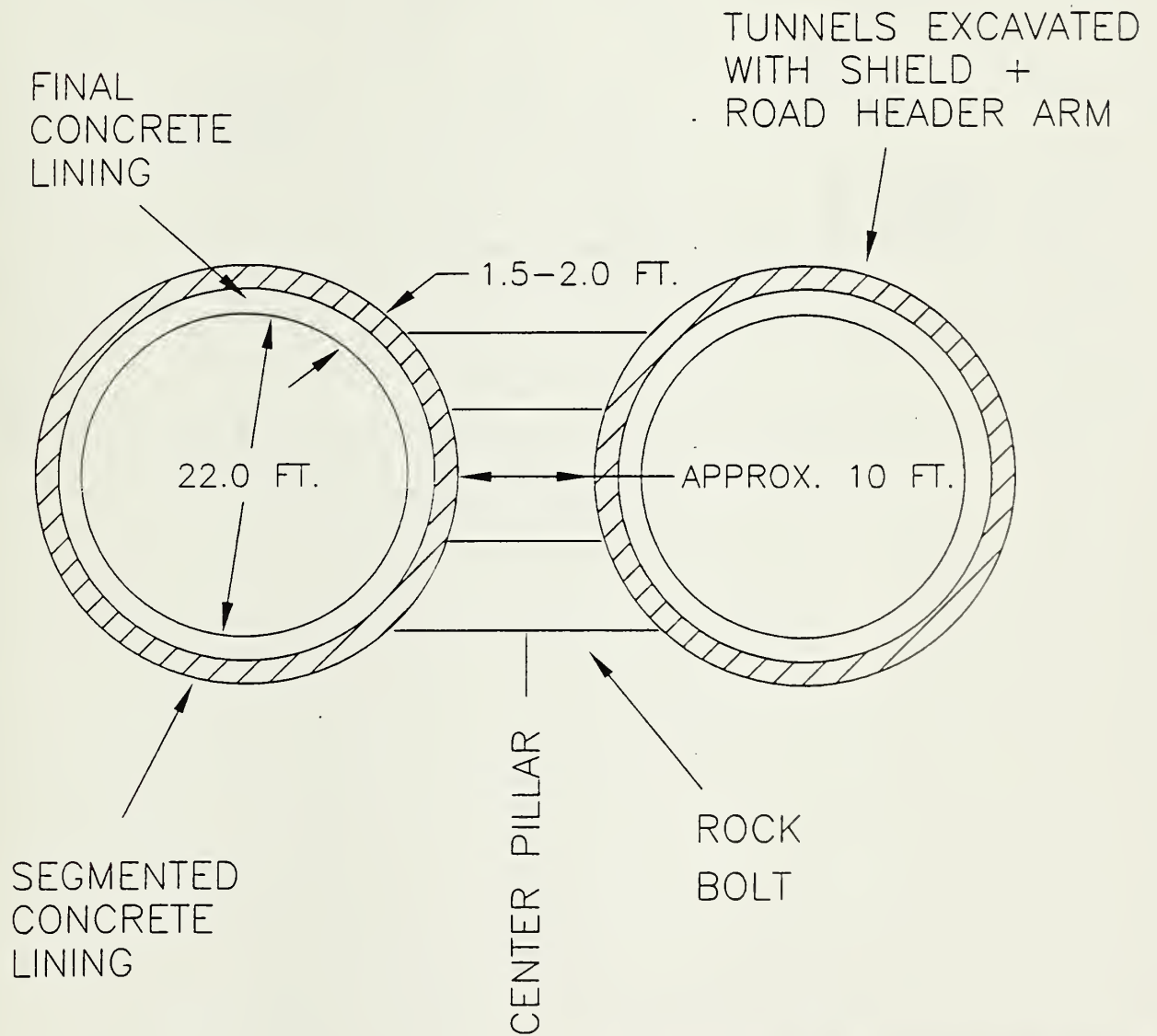


SCHEME B SECTION

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FIGURE 6






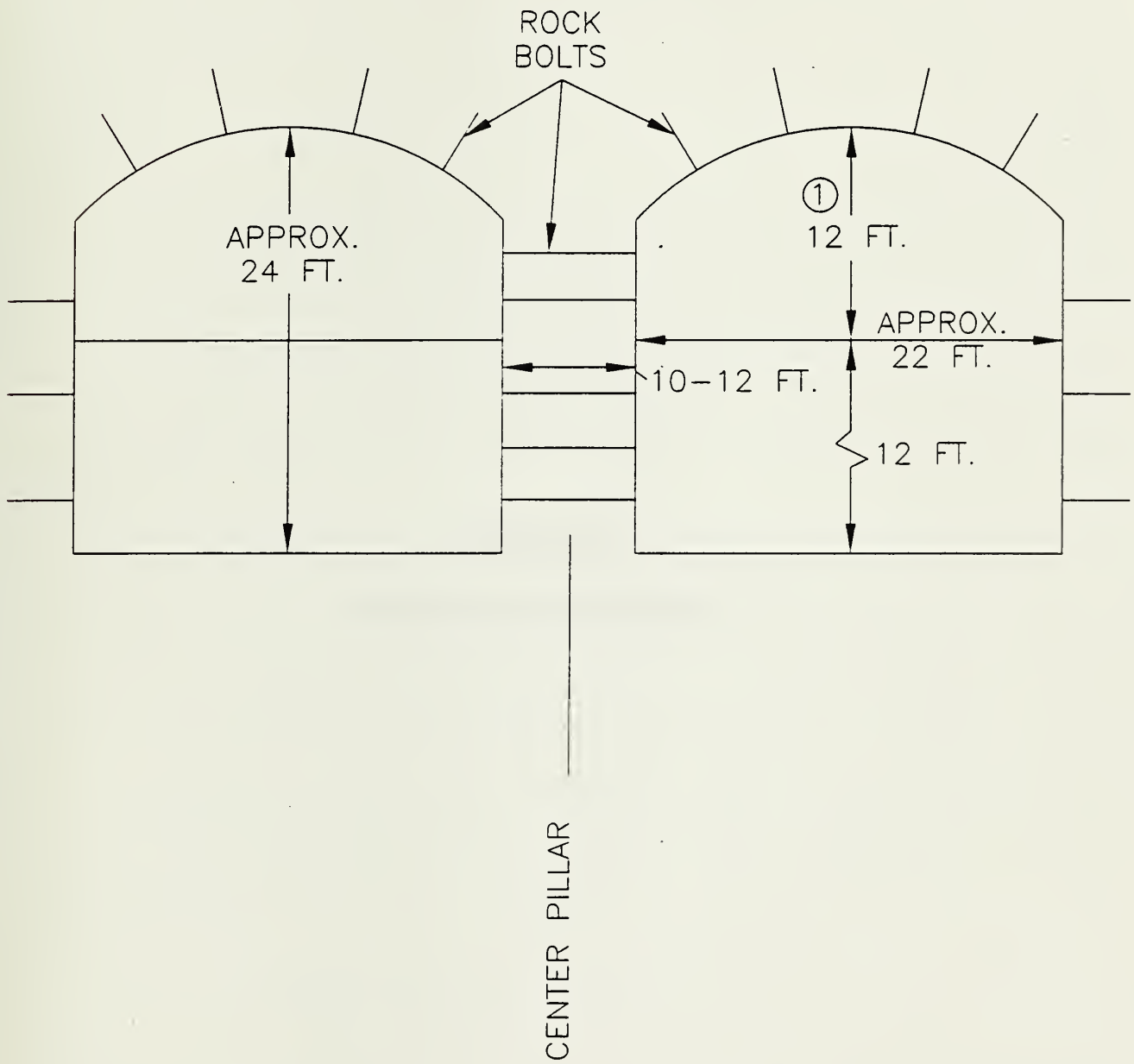
HALL & AIDRICH INC.	
	Geotechnical Engineers & Environmental Consultants
CALTRAIN EIR/EIS TUNNEL CROSS SECTION PROJECT CITY AND STATE	
TWIN BORE SHIELD DRIVEN ALTERNATIVE	
SCALE: NOT TO SCALE	MARCH 1996

FIGURE 7






<b>HALL &amp; VIDRICH INC.</b>	
	Geotechnical Engineers & Environmental Consultants
CALTRAIN EIR/EIS TUNNEL CROSS SECTION PROJECT CITY AND STATE	
TWIN BORE CONVENTIONAL EXCAVATION ALTERNATIVE	
SCALE: NOT TO SCALE	MARCH 1996

FIGURE 8



**APPENDIX A**  
**BASIS OF COST ESTIMATE**



**BASIS OF COST ESTIMATE**  
Caltrain Downtown Extension Project  
Peninsula Corridor Joint Powers Board  
Tunnel Conceptual Cost Estimates  
by  
Haley & Aldrich

## **1.0 GENERAL**

This document is the basis of cost estimate for the following cost estimates. The tunnel construction methods were estimated by Haley & Aldrich as a subcontract to Dames and Moore and to provide assistance to ICF Kaiser Engineers with regards to the subject project. Haley & Aldrich has made many assumptions in preparing these cost estimates as detailed herein and the report entitled "Final Report on Caltrain Downtown Station Relocation". These cost estimates are intended to provide an accurate assessment of costs in 1996 dollars and 1996 market conditions in the San Francisco Bay Area. Additionally, these estimates were prepared for use in the DEIS/DEIR process. Construction cost estimates should be performed during preliminary and final design after more geotechnical information and the schedule are known.

The schematic designs for the three estimates are detailed in Figure 5 for the single tunnel with two drifts, Figure 6 for the Nicholson scheme and Figure 7 for the twin tunnel. It is also suggested that the reader review Nicholson's Report entitled "Caltrain Downtown Extension, San Francisco, CA, Ground Treatment and Support Feasibility Study, Report Prepared for Dames & Moore" for detailed information regarding the cross section shown in Figure 6. These costs are based on ground conditions described by Dames and Moore in their report entitled, "Final Report Geotechnical Site Investigation, CALTRAIN S.F. Downtown Station Relocation EIS/EIR", dated September 25, 1995. These cost estimates only provide conceptual costs for tunneling and initial support related to tunneling. Not included in these cost estimates are costs for shafts, underpinning, final lining and finishes. Muck disposal also is not shown in the estimate. It is believed that some of the muck can be sold and some will be disposed. These muck costs are assumed to cancel for a zero cost to the project.

## **2.0 CONSTRUCTION METHODOLOGY**

### **2.1 General**

This section describes the assumed methodology for the three tunnel construction methods.



## **2.2 Single Tunnel (Two Drifts)**

### **2.2.1 General**

The single tunnel with two drifts is detailed in Figure 5 and has five major cost components:

1. Drive Addits
2. Spile Roof
3. Top Heading Excavation
4. Bench Excavation
5. Mobilization/Demobilization
6. Dewatering

### **2.2.2 Drive Addits**

This item includes anticipated costs for driving wall plate drifts/addits. It is anticipated that these wall plate drifts will be constructed concurrently with one heading being approximately 200 to 300 ft ahead of the second heading. The drifts are expected to be 12 ft wide by 14 ft high, and excavated using road headers and rubber tired vehicles. Initial ground support will be provided by shotcrete and lattice girders. It is assumed that excavation will be performed by utilizing a primary construction shaft near the center of the tunnel. However, multiple shafts and headings could be used. The drifts are also anticipated to facilitate the dewatering of the formation prior to excavation of the top heading.

### **2.2.3 Spile Roof**

This item will be done from the wall plate drifts. It will include not only constructing spiling in the roof section, but also constructing the concrete wall plates, pre-support of the walls for bench excavation and installation of well points to facilitate dewatering for bench excavation. Spiling from the roof will be done from a drill jumbo using jack leg drills. It is anticipated that 6- to 8-ft rods will be installed, coupled to the desired length and grouted in. Spiling will consist of 1-inch steel bars and will be installed on 1-1/2- to 2-ft centers. The pre-support of the bench wall will be performed following the spiling operation. The wall pre-support will consist of 1 inch diameter steel bars installed on 1- to 2-ft centers down through the sidewalls. The pre-support of the bench walls will be followed by the construction of a concrete wall plate which is assumed to be 2 ft thick and 6 to 8 ft high as shown in Figure 5.

### **2.2.4 Top Heading Excavation**

This item will include excavation of the top heading between the two wall plate drifts. The top heading excavation will be performed in one large opening using two road headers. Rubber tired vehicles will be used to carry muck from the heading to the shaft. During the top heading excavation, the linear plant installed during the construction of the addits will continue to be utilized. The construction sequence will include: 1) excavation of the face using road headers; 2) support with steel ribs which



will be founded on the wall drifts and a temporary center support; 3) minor rock bolting as necessary; and 4) fiber shotcrete of the whole opening which will be approximately 300 to 500 ft behind the face excavation.

#### **2.2.5 Bench Excavation**

This item includes excavation of the bench which will also be done with two road headers and rubber tired vehicles. It is assumed that bench excavation will be done sequentially as shown in Figure 5, or it could be done as a full face if desired. Shotcreting of the walls will be accomplished 300 to 500 ft behind the excavation. Minor rockbolting also will be performed as necessary.

#### **2.2.6 Mobilization/Demobilization**

This item includes plant set-up, erection of equipment and portal development or construction of starter tunnels as necessary. Also included in this item is initial dewatering which can be accomplished from the surface. We assumed that tunneling will be part of a larger contract which will include cut and cover construction and costs not specific to tunnel construction. No costs for project start-up were included.

#### **2.2.7 Dewatering**

This item includes costs for installing well points from within addits to dewater the bench prior to excavation. Following the wall plate construction, well points will be installed to facilitate dewatering. It is assumed that a well point will be installed every 20 to 40 ft and connected to a riser system to facilitate dewatering of the bench.

### **2.3 Single Tunnel With Longitudinal Spiling**

#### **2.3.1 General**

The single tunnel with longitudinal spiling is shown in Figure 6 and detailed in the Nicholson Report. It consists of five tasks including:

1. Drive Addits
2. Spile Roof
3. Top Heading Excavation
4. Bench Excavation
5. Mobilization/Demobilization
6. Dewatering

#### **2.3.2 Drive Addit**

This item includes the construction of one center addit. It is anticipated that this addit will be utilized to facilitate dewatering and construction. This addit is assumed to be 12 ft wide by 14 ft high, and will be accomplished using one road header in each heading. It is assumed that excavation will be performed from one center shaft and have a heading in each direction.



### **2.3.3 Spile Roof**

This item includes installation of longitudinal spiling and micro piles as described in Nicholson's report. This longitudinal spiling section is shown in Figure 6. Also included in this item is the list of dewatering wells to be installed down the sides of the top heading after excavation is performed. It is anticipated that a center shaft will be used, with spiling performed in one heading concurrent with top heading excavation in the other heading. The operations will switch headings after each 20 ft advance.

### **2.3.4 Top Heading Excavation**

This item will include excavation of the top heading. This item will be performed concurrently with spile roof construction. It is anticipated that a full face will be excavated using two road headers and rubber tired vehicles in each heading. Support will be done using steel ribs of various sizes as described in the Nicholson report. Minor rock bolting and fiber reinforced shotcreting will be performed 300 to 500 ft behind the face excavation.

### **2.3.5 Bench Excavation**

This item will include bench excavation similar to that described for the single tunnel option with two adds.

### **2.3.6 Mobilization/Demobilization**

This item will include mobilization and demobilization similar to that described for the single tunnel option with two adds.

### **2.3.7 Dewatering**

This item includes installation of well points from the top heading to dewater the bench prior to excavation.

## **2.4 Twin Tunnels**

### **2.4.1 General**

The twin tunnel alternative is detailed in Figure 7 and has five major cost components:

1. Production Tunnel Mining
2. Shaft Support
3. Mobilization /Demobilization
4. Dewatering
5. Cross passages



#### **2.4.2 Production Tunnel Mining**

This item will include the internal tunnel production operations for excavation of the tunnel and installation of initial ground support. It was assumed that two tunnels would be excavated concurrently in opposite directions from the center shaft. The production and shaft support are concurrent operations for the shield-driven tunnels. It is assumed that excavation will be performed using a road header mounted on rails inside the shield. The shield will provide overhead protection, have jack leg drills mounted in the upper haunches for installation of spiling, and have the capacity to erect and expand concrete segments 12 inches thick. It is anticipated that six to eight segments will comprise a 360-degree ring and that these rings will be three to four feet wide. The shield will have jacks which will push off the erected ring for propulsion. Concrete segments, will be used for support, erected by a machine, expanded out against the rock wall and blocked as the shield pushes forward. It is assumed that spiling will be installed in 75 percent of the total tunnel length prior to excavation.

#### **2.4.3 Shaft Support**

This item will include shaft operations necessary to support the twin shield driven tunnels. It is assumed that only two tunnel shield machines will be purchased and used for the excavation of one tunnel in each direction from the central shaft. Shaft operations are therefore assumed to support the two headings concurrently.

#### **2.4.4 Mobilization/ Demobilization**

This item will be similar to mobilization/demobilization for the single tunnel options, except it is anticipated that additional time will be necessary to erect the tunnel shields. Time was also included to re-erect the tunnel shield and for set-up operations in the second tunnel drive.

#### **2.4.5 Dewatering**

In addition to the dewatering which will be performed from the surface, included in the mobilization and demobilization, it is anticipated that additional dewatering will be necessary. This estimate assumed that a directional drill borehole will be constructed from the shaft and an 18-inch slotted PVC casing will be installed in this hole to facilitate dewatering for each of the two tunnels. This water will be pumped to surface drains or city sewers. This construction will be performed prior to initiating the shield tunnel operations.

#### **2.4.6 Cross Passages**

This item includes the construction of five cross passages between the two tunnels. It is assumed that cross passages will be required every 800 ft for the twin tunnel option, thus requiring approximately four passages to cover the length of tunnel which is anticipated to be 3,900 ft. Once excavated, these cross passages are



assumed to be 10 ft wide by 12 ft high and will be supported with shotcrete and lattice girders. They will be constructed after the completion of bench excavation.

### **3.0 PRODUCTION RATES**

The production rates provided in the cost estimates are based on our tunneling experience in the San Francisco and California areas, and our knowledge of subsurface conditions. These rates are average rates encompassing the entire run and include time for a learning curve. These rates do not include time for the portal production which is included in the mobilization/demobilization costs. The rates for the Nicholson scheme are limited to the rate of spiling production, because the operation will be performed concurrently by switching headings.

### **4.0 PRICING**

Pricing was based on our experience and rates for similar projects in the San Francisco area. The labor rates used on similar projects in the San Francisco area include fringes and payroll taxes. The labor rates do not include Worker's Compensation. Worker's Compensation is included in the labor distributables. Overtime was accounted for by adding a 50% premium to the overtime hours. For example, normal operations would be 24 hrs per day comprised of two ten-hour production shifts and one four-hour maintenance shift. A one hour premium is added for the two hours of overtime in each of the two ten-hour shifts giving an equivalent to 22 hours of straight time shown in the estimates. The equipment and temporary materials fees are based on experience and recent projects in the San Francisco area. Pricing assumes that all work would be performed by the prime contractor except for the costs of installing longitudinal spiling which was taken from the Nicholson report and included as a subcontract cost.

### **5.0 MARK-UPS**

The distributable costs cover indirect costs related to labor and equipment like Worker's Compensation, project insurance, small tools, etc. Overhead and profit covers the general overhead and profit for the Contractor including project overhead and all home office support. Contingency has not been included in these cost estimates based on the request of ICF Kaiser Engineers. It is recommended that a contingency of approximately 25 percent be used for both of the single tunnel options and a contingency of 35 percent be used for the twin tunnel option. These contingency percentages include risk factors for geologic and project unknowns, assumed hydrocarbon hazardous waste conditions along 10 percent of the alignment and the general contingency carried by the tunnel contractors.



## 6.0 CLOSING

This cost estimate, as stated before, was prepared based on conceptual designs and preliminary geotechnical information. Additional cost estimating must be performed during preliminary engineering and final design when better information is available regarding the schedule and geotechnical conditions. These estimates are intended to provide an accurate cost, but their primary purpose is for use in the EIS/EIR process.







CAL Train  
SINGLE TUNNEL  
CONCEPTUAL CONSTRUCTION COST ESTIMATE

DATE 13-May-08

LABOR

Item No	CSI Code	1.) Drive Admit					2.) Spike Roof & Pre Support Walls					3.) Top Heading Excavation					4.) Bench Excavation				
		Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day
1	0	2	Equipment Operator	40.00	22	\$1,760.00	3	Equipment Operator	38.00	22	\$2,508.00	3	Equipment Operator	40.00	22	\$2,640.00	1	Equipment Operator	40.00	22	\$880.00
2	0	9	Miner	32.00	22	\$8,336.00	3	Shifter	33.00	22	\$2,178.00	19	Miner	32.00	22	\$13,376.00	11	Miner	32.00	22	\$7,744.00
3	0	3	Shifter	33.00	22	\$2,178.00	6	Miner	32.00	22	\$5,632.00	5	Shifter	32.00	22	\$3,630.00	4	Shifter	33.00	22	\$2,904.00
4	0	4	Dump Truck Operator	33.50	22	\$2,948.00	2	STV Operators	33.50	22	\$1,474.00	3	Dump Truck Operator	33.50	22	\$2,211.00	3	Dump Truck Operator	33.50	22	\$2,211.00
5	0	1	Walker	35.00	28	\$910.00	2	WALL SUPPORT CREW			\$0.00	1	Walker	35.00	28	\$910.00	1	Walker	35.00	28	\$910.00
6	0	1	Mechanic	39.00	28	\$1,014.00	2	Drill Operator	34.00	22	\$1,466.00	1	Mechanic	39.00	28	\$1,014.00	1	Mechanic	39.00	28	\$1,014.00
7	0	1	Electrician	35.00	28	\$910.00	4	Miner	32.00	22	\$5,632.00	1	Electrician	35.00	28	\$910.00	1	Electrician	35.00	28	\$910.00
8	0	1	Crane Operator	40.00	22	\$880.00	1	Walker	35.00	28	\$910.00	1	Crane Operator	40.00	22	\$880.00	1	Crane Operator	40.00	22	\$880.00
9	0	1	Olser	34.00	28	\$884.00		CONCRETE CREW			\$0.00	1	Olser	34.00	28	\$884.00	1	Olser	34.00	28	\$884.00
10	0	1	Top Lander	31.00	22	\$682.00	10	Miner	32.00	22	\$7,040.00	1	Top Lander	31.00	22	\$682.00	1	Top Lander	31.00	22	\$682.00
11	0	1	Bottom Lander	31.00	22	\$682.00	2	Shifter	33.00	22	\$1,452.00	1	Bottom Lander	31.00	22	\$682.00	1	Bottom Lander	31.00	22	\$682.00
12	0	1	Warehouseman (miner)	32.00	22	\$704.00	2	Mechanic	39.00	28	\$2,028.00	1	Warehouseman (miner)	32.00	22	\$704.00	1	Warehouseman (miner)	32.00	22	\$704.00
		1	Picker Operator	38.80	22	\$649.20						1	Picker Operator	38.80	22	\$649.20	1	Picker Operator	38.80	22	\$649.20
		1	Teamster	33.00	22	\$726.00						1	Teamster	33.00	22	\$726.00	1	Teamster	33.00	22	\$726.00
												2	Driller	34.00	22	\$1,466.00	2	Driller	34.00	22	\$1,466.00
												1	Pump Operator	33.50	22	\$737.00	1	Pump Operator	33.50	22	\$737.00
												1	STV Operator	33.50	22	\$737.00	1	STV Operator	33.50	22	\$737.00

TOTAL		\$21,463.20		TOTAL		\$30,350.00		TOTAL		\$33,068.20		TOTAL		\$24,950.20
Units Per Day	18			Units Per Day	20			Units Per Day	40			Units Per Day	40	
Dollars per Unit	\$1,341.45			Dollars per Unit	\$1,517.50			Dollars per Unit	\$828.71			Dollars per Unit	\$623.76	
Hours per Unit	36.50			Hours per Unit	45.70			Hours per Unit	24.60			Hours per Unit	18.55	

Item No	CSI Code	5.) Mob/ Demob					6.) Dewatering					7.)					8.)				
		Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day
1	0	2	Equipment Operator	40.00	22	\$1,760.00		DEWATERING WELL INSTALLATION													
2	0	4	Miner	32.00	22	\$2,616.00	2	Drill Operator	34.00	22	\$1,466.00										
3	0	3	Shifter	33.00	22	\$2,178.00	7	Miner	32.00	22	\$4,928.00										
4	0	2	Dump Truck Operator	32.50	22	\$1,474.00	1	Shifter	33.00	22	\$726.00										
5	0	1	Walker	35.00	28	\$910.00															
6	0	1	Mechanic	39.00	28	\$1,014.00															
7	0	1	Electrician	35.00	28	\$910.00															
8	0	1	Crane Operator	40.00	22	\$880.00															
9	0	1	Olser	34.00	28	\$884.00															
10	0	1	Top Lander	31.00	22	\$682.00															
11	0	1	Bottom Lander	31.00	22	\$682.00															
12	0	1	Warehouseman (miner)	32.00	22	\$704.00															
		1	Picker Operator	38.80	22	\$649.20															
		1	Teamster	32.00	22	\$726.00															

TOTAL		\$16,469.20		TOTAL		\$7,150.00		TOTAL		50.00		TOTAL		50.00
Units Per Day	0.02			Units Per Day	20			Units Per Day	1			Units Per Day	1	
Dollars per Unit	\$823,480.00			Dollars per Unit	\$357.50			Dollars per Unit	50.00			Dollars per Unit	50.00	
Hours per Unit	23,900.00			Hours per Unit	11.00			Hours per Unit	0.00			Hours per Unit	0.00	



CAL Train  
SINGLE TUNNEL  
CONSTRUCTION COSTS

13-May-08

## EQUIPMENT

Item No	CSI Code	1.) Drive Adit				2.) Spoil Roof & Pile Support Walls				3.) Top Heading Excavation				4.) Bench Excavation			
		Quantity	Designation	Rate \$/Day	Cost \$/Day	Quantity	Designation	Rate \$/Day	Cost \$/Day	Quantity	Designation	Rate \$/Day	Cost \$/Day	Quantity	Designation	Rate \$/Day	Cost \$/Day
1		2	Roadheader	2600	5200	4	Drill Jumbo	900	3600	2	Roadheader	2600	5200	2	Roadheader	2600	5200
2		4	STS Muckers	1000	4000	1	Compressor	400	400	4	STS Muckers	1000	4000	4	STS Muckers	1000	4000
3		2	100 gpm Pumps	100	200	2	100 gpm Pumps	150	300	2	100 gpm Pumps	100	200	2	100 gpm Pumps	100	200
4		2	100 HP Fans	150	300	4	STS Mucker	1000	4000	2	100 HP Fans	150	300	2	100 HP Fans	150	300
5		2	Inert plant		0	4	Jack Leg	35	140	2	Inert plant		0	2	Inert plant		0
6		2	Compressors	400	800		CONCRETE			2	Compressors	400	800	2	Compressors	400	800
		2	Shotcrete Plant	400	800	1	Concrete Pump	175	175	2	Shotcrete Plant	400	800	2	Shotcrete Plant	400	800
		2	Driller	35	70	WALL SUPPORT CREW				2	Driller	35	70	2	Driller	35	70
		2	Grout Pump	175	350	2	Drill Rig	100	200	2	Grout Pump	175	350	2	Grout Pump	175	350
		1	100 Ton Crane	2200	2200					1	100 Ton Crane	2200	2200	1	100 Ton Crane	2200	2200
		2	Front End Loader	780	1520					2	Front End Loader	780	1520	2	Front End Loader	780	1520
		1	Muck Box 10 yd	140	140					1	Muck Box 10 yd	140	140	1	Muck Box 10 yd	140	140
		6	Light Plant	120	720					6	Light Plant	120	720	6	Light Plant	120	720
		1	Man Cage	150	150					1	Man Cage	150	150	1	Man Cage	150	150
		1	Rocker	150	150					1	Rocker	150	150	1	Rocker	150	150
		1	Flatbed	26	26					1	Flatbed	26	26	1	Flatbed	26	26
		2	Jack Leg Drills	35	70					2	Jack Leg Drills	35	70	2	Jack Leg Drills	35	70
TOTAL					\$16,696.00	TOTAL			\$6,815.00	TOTAL			\$16,696.00	TOTAL			\$16,696.00
Units per Day					18	Units per Day			20	Units per Day			40	Units per Day			40
Dollars per Unit					\$1,043.50	Dollars per Unit			\$440.75	Dollars per Unit			\$417.40	Dollars per Unit			\$417.40

5.) Mob/ Demob					6.) Dewatering									
Quantity	Designation	Rate \$/Day	Cost \$/Day		Quantity	Designation	Rate \$/Day	Cost \$/Day						
2	Roadheader	2600	5200		2	Drill Rig	500	1000						
4	STS Muckers	1000	4000		2	100 gpm Pumps	100	200						
2	100 gpm Pumps	100	200											
2	100 HP Fans	150	300											
2	Inert plant		0											
2	Compressors	400	800											
2	Shotcrete Plant	400	800											
2	Driller	35	70											
2	Grout Pump	175	350											
1	100 Ton Crane	2200	2200											
2	Front End Loader	780	1520											
1	Muck Box 10 yd	140	140											
6	Light Plant	120	720											
1	Man Cage	150	150											
1	Rocker	150	150											
1	Flatbed	26	26											
2	Jack Leg Drills	35	70											
TOTAL			\$16,696.00		TOTAL			\$1,200.00						
Units per Day			0.02		Units per Day			20						
Dollars per Unit			\$834,800.00		Dollars per Unit			\$60.00						



CAL Tran  
SINGLE TUNNEL  
CONCEPTUAL CONSTRUCTION COST ESTIMATE  
0  
DATE 13-May-86

TEMPORARY MATERIAL

TEMPORARY MATERIAL																						
1.) Drive Adit							2.) Spile Roof & Pre Support Walls					3.) Top Heading Excavation					4.) Bench Excavation					
Item No.	CSI Code	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	
1		2	lineal plant	ft	50	100	300	Spiling	lbs/ft	1.25	375	0.8	Shotcrete	cy/ft	90	72		0.7	Shotcrete	cy/ft	90	63
2		1.1	Shotcrete	cy/ft	90	99	1.2	Concrete	cy/ft	80	96	2150	Steel Support- 50 lbs	lbs/ft	0.9	1,935		1	Miscellaneous	ft	50	50
3		1	Lattice girders	ft	70	70	8	Well Points	FT/ft	10	80	1	Miscellaneous	ft	50	50						0
4					0	0					0				0	0					0	
5					0	0					0				0	0					0	
6					0	0					0				0	0					0	
7					0	0					0				0	0					0	
8					0	0					0				0	0					0	
			Total per ft			289		Total per ft			531		Total per ft			2,057		Total per ft				113

		5.) Mobil Demoo					6.) Dewatering														
Item No.	CSI Code	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost
							8	Well Points	FT/ft	10	80										
			Total per ft			0		Total per ft			80										



## SUMMARY

SUMMARY															
Item No	CSI Code	Description	Quantity	Unit	Labor Dollars	Subcontract	Temporary Material Dollars	Const. Equipment Dollars	Total Direct Dollars	Hours	Labor \$/Unit	Subcontract \$/Unit	Temp. Mat. \$/Unit	Const Equip \$/Unit	Total Unit Price \$/Unit
1		Drive Adidt	3,900	FT	3,794,505		581,100	2,359,744	6,735,348.75	111,150	972.95		149.00	905.08	1,727.01
2		Sple Roof	3,900	FT	3,512,925		2,091,000	1,133,925	7,337,850.00	99,450	900.75		890.00	290.75	1,861.50
3		Top Heading Excavation	3,900	FT	6,448,299		8,080,800	4,367,220	18,896,319.00	191,880	1,653.41		2,072.80	1,119.80	4,845.21
4		Bench Excavation	3,900	FT	2,432,845		440,700	1,827,860	4,501,204.50	72,345	623.78		113.00	417.40	1,154.18
5		Mob/Demob	1	EA	823,480		0	824,800	1,858,280.00	23,900	823,480.00		0.00	834,800.00	1,858,280.00
6		Dewatering	3,900	FT	1,394,250		234,000	234,000	1,862,250.00	42,900	357.50		60.00	60.00	477.50
7															
8															
TOTAL DIRECT COST						18,405,064	0	12,027,665	10,557,549	40,991,237					
Construction Distributables - Labor					1,472,487		8.00% Percent								
Construction Distributables - Materials					841,932		7.00% Percent								
SUBTOTAL - Labor & Matl Distr					2,314,419										
Subcontractor's Overhead & Profit					N/A		0.00% Percent								
Overhead and Profit					10,828,413		25.00% Percent								
SUBTOTAL - Overhead & Profit					10,828,413										
Escalation					0			Percent							
SUBTOTAL - Escalation					0										
Contingency					0		NA	Percent							
SUBTOTAL - Contingency					0										
TOTAL					54,132,064										



CAL Train  
SINGLE TUNNEL w/ Longitudinal Spiling  
CONCEPTUAL CONSTRUCTION COST ESTIMATE

DATE 13-May-08

L A B O R

1.) Drive Assist						2.) Spile Roof & Pre Support Walls						3.) Top Heading Excavation						4.) Bench Excavation						
Item No.	CSI Code	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day			
1	0	1	Equipment Operator	40.00	22	\$880.00	1	Equipment Operator	36.00	22	\$838.00	3	Equipment Operator	40.00	22	\$2,640.00	1	Equipment Operator	40.00	22	\$880.00			
2	0	8	Miner	32.00	22	\$3,520.00	2	Shifter	33.00	22	\$1,452.00	19	Miner	32.00	22	\$13,378.00	11	Miner	32.00	22	\$7,744.00			
3	0	2	Shifter	33.00	22	\$1,452.00	12	Miner	32.00	22	\$8,448.00	5	Shifter	33.00	22	\$3,630.00	4	Shifter	33.00	22	\$2,904.00			
4	0	2	Dump Truck Operator	33.50	22	\$1,474.00	1	STV Operators	33.50	22	\$737.00	3	Dump Truck Operator	33.50	22	\$2,211.00	3	Dump Truck Operator	33.50	22	\$2,211.00			
5	0	1	Walker	35.00	26	\$910.00	1	Mechanic	39.00	26	\$1,014.00	1	Walker	35.00	26	\$910.00	1	Walker	35.00	26	\$910.00			
6	0	1	Mechanic	39.00	26	\$1,014.00	6	Ortl Operator	34.00	22	\$4,488.00	1	Mechanic	39.00	26	\$1,014.00	1	Mechanic	39.00	26	\$1,014.00			
7	0	1	Electrician	35.00	26	\$910.00	1	Walker	35.00	26	\$900.00	1	Electrician	35.00	26	\$910.00	1	Electrician	35.00	26	\$910.00			
8	0	1	Crane Operator	40.00	22	\$880.00	1	Ground Support Engineer	40.00	26	\$1,040.00	1	Crane Operator	40.00	22	\$880.00	1	Crane Operator	40.00	22	\$880.00			
9	0	1	Olser	34.00	26	\$884.00						1	Olser	34.00	26	\$884.00	1	Olser	34.00	26	\$884.00			
10	0	1	Top Lander	31.00	22	\$682.00						1	Top Lander	31.00	22	\$682.00	1	Top Lander	31.00	22	\$682.00			
11	0	1	Bottom Lander	31.00	22	\$682.00						1	Bottom Lander	31.00	22	\$682.00	1	Bottom Lander	31.00	22	\$682.00			
12	0	1	Warehouseman (miner)	32.00	22	\$704.00						1	Warehouseman (miner)	32.00	22	\$704.00	1	Warehouseman (miner)	32.00	22	\$704.00			
		1	Picker Operator	36.60	22	\$849.20						1	Picker Operator	36.60	22	\$849.20	1	Picker Operator	36.60	22	\$849.20			
		1	Teamster	33.00	22	\$726.00						1	Teamster	33.00	22	\$726.00	1	Teamster	33.00	22	\$726.00			
												2	Driller	34.00	22	\$1,496.00	2	Driller	34.00	22	\$1,496.00			
												1	Pump Operator	33.50	22	\$737.00	1	Pump Operator	33.50	22	\$737.00			
												1	STV Operator	33.50	22	\$737.00	1	STV Operator	33.50	22	\$737.00			
TOTAL						\$15,567.20	TOTAL						TOTAL						TOTAL					
Units Per Day						18	Units Per Day						Units Per Day						Units Per Day					
Dollars per Unit						\$872.95	Dollars per Unit						Dollars per Unit						Dollars per Unit					
Hours per Unit						26.50	Hours per Unit						Hours per Unit						Hours per Unit					

Item No	CSI Code	5.) Mob/ Demob					6.) Dewatering					7.)					8.)										
		Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day						
0	2	Equipment Operator	40.00		22	\$1,760.00		DEWATERING WELL INSTALLATION																			
0	4	Miner	32.00		22	\$2,816.00	2	Drill Operator	34.00		22	\$1,496.00															
0	3	Shifter	33.00		22	\$2,178.00	7	Miner	32.00		22	\$4,928.00															
0	2	Dump Truck Operator	33.50		22	\$1,474.00	1	Shifter	33.00		22	\$726.00															
0	1	Walker	35.00		26	\$910.00																					
0	1	Mechanic	39.00		26	\$1,014.00																					
0	1	Electrician	35.00		26	\$910.00																					
	1	Crane Operator	40.00		22	\$880.00																					
	1	Olser	34.00		26	\$884.00																					
	1	Top Lander	31.00		22	\$682.00																					
	1	Bottom Lander	31.00		22	\$682.00																					
	1	Warehouseman (miner)	32.00		22	\$704.00																					
	1	Picker Operator	36.60		22	\$849.20																					
	1	Teamster	33.00		22	\$726.00																					
TOTAL						\$18,460.20	TOTAL						\$7,150.00	TOTAL						\$0.00	TOTAL						\$0.00
Units Per Day						0.02	Units Per Day						20	Units Per Day						50	Units Per Day						40
Dollars per Unit						\$623,460.00	Dollars per Unit						\$357.50	Dollars per Unit						\$0.00	Dollars per Unit						\$0.00
Hours per Unit						23,900.00	Hours per Unit						11.00	Hours per Unit						0.00	Hours per Unit						0.00



CAL Train  
SINGLE TUNNEL w/ Longitudinal Spiling  
CONSTRUCTION COSTS

13-May-85

## EQUIPMENT

EQUIPMENT		1.) Drive Adit				2.) Spile Roof & Pre Support Walls				3.) Top Heading Excavation				4.) Bench Excavation							
Item No	CSI Code	Quantity	Description	Rate \$/Day	Cost \$/Day	Quantity	Description	Rate \$/Day	Cost \$/Day	Quantity	Description	Rate \$/Day	Cost \$/Day	Quantity	Description	Rate \$/Day	Cost \$/Day				
1		1	Roadheader	2800	2800	4	Drill Jumbo	900	3600	4	Roadheader	2800	10400	2	Roadheader	2800	5200				
2		2	STS Muckers	1000	2000	1	Compressor	400	400	4	STS Muckers	1000	4000	4	STS Muckers	1000	4000				
3		1	100 gpm Pumps	100	100	2	100 HP Fans	150	300	4	100 gpm Pumps	100	400	2	100 gpm Pumps	100	200				
4		1	100 HP Fans	150	150	1	STS Mucker	1000	1000	4	100 HP Fans	150	600	2	100 HP Fans	150	300				
5		1	Inseal plant		0	4	Jack Leg Drill	35	140	2	Inseal plant		0	2	Inseal plant		0				
6		1	Compressors	400	400	1	Concrete Pump	175	175	2	Compressors	400	800	2	Compressors	400	800				
		1	Shotcrete Plant	400	400	2	Drill Rig	100	200	2	Shotcrete Plant	400	800	2	Shotcrete Plant	400	800				
		1	Driller	35	35					2	Driller	35	70	2	Driller	35	70				
		1	Grout Pump	175	175					2	Grout Pump	175	350	2	Grout Pump	175	350				
		1	100 Ton Crane	2200	2200					1	100 Ton Crane	2200	2200	1	100 Ton Crane	2200	2200				
		1	Front End Loader	780	780					2	Front End Loader	780	1520	2	Front End Loader	780	1520				
		1	Muck Box 10 yd	140	140					1	Muck Box 10 yd	140	140	1	Muck Box 10 yd	140	140				
		3	Light Plant	120	360					6	Light Plant	120	720	6	Light Plant	120	720				
		1	Man Cage	150	150					1	Man Cage	150	150	1	Man Cage	150	150				
		1	Rockier	150	150					1	Rockier	150	150	1	Rockier	150	150				
		1	Flatbed	25	25					1	Flatbed	25	25	1	Flatbed	25	25				
		1	Jack Leg Drills	35	35					2	Jack Leg Drill	35	70	2	Jack Leg Drills	35	70				
TOTAL					\$9,661.00	TOTAL					\$5,815.00	TOTAL					\$22,396.00	TOTAL			\$16,696.00
Units per Day					16	Units per Day					20	Units per Day					20	Units per Day			40
Dollars per Unit					\$605.06	Dollars per Unit					\$290.75	Dollars per Unit					\$1,119.80	Dollars per Unit			\$417.40

		5.) Mob/Demo				6.) Dewatering					
Quantity	Designation	Rate \$/Day	Cost \$/Day	Quantity	Designation	Rate \$/Day	Cost \$/Day				
2	Roadheader	2800	5200	2	Drill Rig	500	1000				
4	STS Muckers	1000	4000	2	100 gpm Pumps	100	200				
2	100 gpm Pumps	100	200								
2	100 HP Fans	150	300								
2	Inseal plant		0								
2	Compressors	400	800								
2	Shotcrete Plant	400	800								
2	Driller	35	70								
2	Grout Pump	175	350								
1	100 Ton Crane	2200	2200								
2	Front End Loader	780	1520								
1	Muck Box 10 yd	140	140								
6	Light Plant	120	720								
1	Man Cage	150	150								
1	Rockier	150	150								
1	Flatbed	25	25								
2	Jack Leg Drills	35	70								
T O T A L		\$16,696.00		T O T A L		\$1,200.00					
Units per Day		0.02		Units per Day		20					
Dollars per Unit		\$834,800.00		Dollars per Unit		\$60.00					



CAL Train  
SINGLE TUNNEL w/ Longitudinal Spring  
CONCEPTUAL CONSTRUCTION COST ESTIMATE  
0  
DATE 13-May-08

TEMPORARY MATERIAL

		1.) Drive Addit			2.) Spine Roof & Pile Support Walls			3.) Top Heading Excavation			4.) Bench Excavation					
Item No	CSI Code	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost
1		1	lineal plant	ft	50	50	62	Pipe 4.5 in dia	ft/ft	10	620	0.6	Shotcrete	cy/ft	90	72
2		0.6	Shotcrete	cy/ft	90	54	6	Micro piles	ft/ft	10	60	2150	Steel Support- 50 lbs	lbs/ft	0.9	1,935
3		1	Lattice girders	ft	45	45	1	Miscellaneous	ft/ft	10	10	1	Miscellaneous	ft	15	15
4						0					0	1	lineal plant	ft	50	50
5						0					0					0
6						0					0					0
7						0					0					0
8						0					0					0
			Total per ft			149		Total per ft			690		Total per ft			2,072
																113

5.) Moor Demos						6.) Dewatering											
Item No	CSI Code	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	
							6	Well Points	FT/ft	10	60						
			Total per ft			7,799		Total per ft			60						



DATE 13-May-96

## SUMMARY

[illegible]



CAL Train  
TWIN Tunnels - Shield Driven  
CONCEPTUAL CONSTRUCTION COST ESTIMATE

DATE 8 April 96

LABOR

		1. Production					2. SHAFT					3. MOB/D&MOB				
Item No.	CSI Code	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day
1	0	2	Machine Operator	36.6	24	\$1,652.80	1	Crane Operator	40.00	22	\$860.00	3	Machine Operator	36.6	11	\$1,273.60
2	0	2	Mechanic	35.3	26	\$1,835.60	1	Oiler	34.00	26	\$884.00	6	Mechanic	35.3	11	\$2,329.60
3	0	2	Electrician	34	22	\$1,496.00	1	Top Lander	31.00	22	\$662.00	4	Electrician	34	11	\$1,496.00
4	0	12	Miners	32.00	22	\$6,448.00	1	Bottom Lander	31.00	22	\$682.00	10	Miners	32.00	11	\$3,520.00
5	0	2	Shifter	33.00	26	\$1,716.00	1	Warehouseman (miner)	32.00	26	\$632.00	2	Shifter	33.00	11	\$726.00
6	0	4	Loc Operator	35.30	22	\$3,106.40	2	Pickar Operator	36.60	22	\$1,698.40	4	Loc Operator	35.30	11	\$1,553.20
7	0		BULLGANG		22	\$0.00	2	Wheel Loader Operator	37.60	22	\$1,654.40	1	Crane Operator	40.00	11	\$440.00
8											\$0.00	1	Oiler	34.00	11	\$374.00
9		6	Miners	32.00	22	\$4,224.00					\$0.00	1	Top Lander	31.00	11	\$341.00
10		2	Shifters	33.00	22	\$1,452.00						1	Bottom Lander	31.00	11	\$341.00
11		1	Walker	36.00	26	\$968.00						1	Warehouseman (min)	32.00	11	\$352.00
12												1	Pickar Operator	36.60	11	\$424.60
												1	Walker	36.00	26	\$968.00
												2	Wheel Loader Opera	37.60	11	\$827.20
		TOTAL			234	\$25,118.80	TOTAL		244.20	162	\$7,312.60	TOTAL		490.40	169	\$14,986.60
		Units Per Day			24		Units Per Day			24		Units Per Day				0.004
		Dollars per Unit				\$1,048.62	Dollars per Unit				\$304.70	Dollars per Unit				\$3,748,650.00
		Hours per Unit				31.25	Hours per Unit				8.56	Hours per Unit				106,250.00

		4. Dewatering					5. Cross Passage				
		Quantity	Designation	Rate/Ea	Hrs/day	\$/day	Quantity	Designation	Rate/Ea	Hrs/day	\$/day
		1	Machine Operator	36.6	26	\$1,003.60	1	Machine Operator	36.6	26	\$1,003.60
		1	Mechanic	35.3	26	\$917.80	1	Mechanic	35.3	26	\$917.80
		1	Electrician	34	26	\$884.00	1	Electrician	34	26	\$884.00
		6	Miners	32.00	22	\$4,224.00	4	Miners	32.00	22	\$2,816.00
		1	Shifter	33.00	22	\$726.00	1	Shifter	33.00	22	\$726.00
		0	Loc Operator	35.30		\$0.00	1	Loc Operator	35.30	22	\$776.60
		1	Crane Operator	40.00	22	\$860.00	1	Crane Operator	40.00	22	\$860.00
		0	Oiler	34.00	22	\$748.00	1	Oiler	34.00	22	\$746.00
		0	Top Lander	31.00		\$0.00	0	Top Lander	31.00	22	\$0.00
		0	Bottom Lander	31.00		\$0.00	0	Bottom Lander	31.00	22	\$0.00
		0	Warehouseman (min)	32.00		\$0.00	0	Warehouseman (miner)	32.00	22	\$0.00
		1	Pickar Operator	36.60		\$0.00	1	Pickar Operator	36.60	22	\$849.20
		1	Walker	36.00	26	\$968.00	1	Walker	36.00	26	\$968.00
		1	Wheel Loader Opera	37.60	22	\$827.20	1	Wheel Loader Operator	37.60	22	\$827.20
		TOTAL		490.40	214	\$11,198.60	TOTAL		490.40	324	\$11,416.40
		Units Per Day			50		Units Per Day			2	
		Dollars per Unit				\$223.97	Dollars per Unit				\$5,708.20
		Hours per Unit				6.48	Hours per Unit				162.00



CAL Train  
TWIN Tunnels - Shield Driven  
CONCEPTUAL CONSTRUCTION COST ESTIMATE

13-May-96

## EQUIPMENT

Item No.	CSI Code	1. PRODUCTION				2. SHAFT				3. MOB/ DeMOB				4. Dewatering			
		Quantity	Designation	Rate \$/Day	Cost \$/Day	Quantity	Designation	Rate \$/Day	Cost \$/Day	Quantity	Designation	Rate \$/Day	Cost \$/Day	Quantity	Designation	Rate \$/Day	Cost \$/Day
1		2	Shield Machine	3480	6960	1	100 Ton Crane	2200	2200	2	Shield Machine	2700	5400	1	Directional Drill	2300	2300
2		4	Locl	690	2760	2	Pickler	1000	2000	4	Locl	690	2760	1	500 gpm pump	500	500
3		2	Liner Plant	48	96	1	Flatbed	200	200	2	Jack leg drill	35	70	1	Jack leg drill	35	35
4		2	200 Hp Fan	65	130	2	Front End Loader	760	1520	2	200 Hp Fan	65	130	1	Control Trailer	2000	2000
5		2	Grout Pump	250	500					2	Grout Pump	250	500	1	Grout Pump	250	250
6		2	Compressor	500	1000					2	Compressor	400	800	1	Compressor	400	400
		2	Jack leg drill	35	70					1	100 Ton Crane	2200	2200	1	100 Ton Crane	2200	2200
										1	Pickler	1000	1000	1	Pickler	1000	1000
										1	Flatbed	200	200	1	Flatbed	200	200
		TOTAL		\$11,516.00		TOTAL		\$5,920.00		TOTAL		\$13,060.00		TOTAL		\$8,885.00	
		Units per Day		24		Units per Day		24		Units per Day		0.004		Units per Day		50	
		Dollars per Unit		\$479.83		Dollars per Unit		\$246.67		Dollars per Unit		\$3,265,000.00		Dollars per Unit		\$177.70	

5.) Cross Passage							
Quantity	Designation	Rate \$/Day	Cost \$/Day				
1	Rooshaader	2200	2200				
2	Locl	690	1380				
1	Jack leg drill	35	35				
1	200 Hp Fan	65	65				
1	Grout Pump	250	250				
1	Compressor	400	400				
1	100 Ton Crane	2200	2200				
1	Pickler	1000	1000				
1	Flatbed	200	200				
TOTAL		\$7,730.00					
Units per Day		2					
Dollars per Unit		\$3,865.00					



CAL Train  
TWIN Tunnels - Shield Driven  
CONCEPTUAL CONSTRUCTION COST ESTIMATE  
0  
DATE 13-May-98

TEMPORARY MATERIAL

Item No.	CSI Code	1. Production					2. SHAFT					3. MOB/DeMOB					4. Dewatering				
		Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost	Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost
1		100	Spiling	lbs/ft	1.25	125				0				0			1	18 inch Casing	FVFI	50	50
2		2.8	Concrete Segments	cy/ft	400	1,120				0				0			1	Miscellaneous	ft/ft	25	25
3		1	Linear Plant	ft	48	48				0				0			1	Linear Plant	ft/ft	35	35
4		1	Miscellaneous	ft	50	50				0				0							0
5						0				0				0							0
6						0				0				0							0
7						0				0				0							0
8						0				0				0							0
			Total per ft			1,343												Total per ft			110

Item No.	CSI Code	5. Cross Passage				
		Quantity	Description	Material Unit	Cost per Mat Unit	Material Cost
		1	Shotcrete	cy/ft	90	90
		1	Lattice girders	ft	50	50
			Total per ft			140



## **APPENDIX C**

**REPORT BY NICHOLSON CONSTRUCTION COMPANY:  
"GROUND TREATMENT AND SUPPORT FEASIBILITY STUDY"**



**CALTRAIN  
DOWNTOWN EXTENSION  
SAN FRANCISCO, CA**

**Ground Treatment and Support Feasibility Study**

Report Prepared by

NICHOLSON CONSTRUCTION COMPANY

for

DAMES&MOORE

Report No. S9612-03  
April 1996



May 6, 1996

Nicholson Construction Company

Dames & Moore  
221 Main Street, Suite 600  
San Francisco, CA 94105

2140 W. Winton Avenue  
Hayward, California 94545  
Telephone: 510.785.6222  
Facsimile: 510.785.6247

Attn.: Mr. Demetrious Koutsoftas  
Principal

Our ref. no.: S9612-03

Subject: Caltrain Downtown Extension

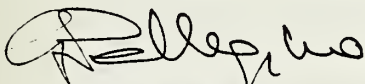
Dear Mr. Koutsoftas:

Please find enclosed two copies of our Report No. S9612-03 "Ground Treatment and Support Feasibility Study". The Report includes the modifications discussed with you as well as our cost assessment. Several case histories have been attached to the report.

We are always available to meet with you and your Client to further discuss the various options considered in the Report.

Should you need more informations or have any questions please do not hesitate to contact me

Sincerely,



Dr. Guido Pellegrino

Enclosures

c.c.: file S9612



CALTRAIN  
DOWNTOWN EXTENSION  
SAN FRANCISCO, CA

Ground Treatment and Support Feasibility Study

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## **Ground Treatment and Support Feasibility Study CALTRAIN SAN FRANCISCO DOWNTOWN EXTENSION**

Report No. S9612-03  
April 1996

### **1. INTRODUCTION**

This report presents the ground treatment and support feasibility study for the above referenced Project. An evaluation of the available information is done in order to determine the most appropriate method of stabilizing the ground surrounding the proposed tunnel.

Final recommendations on the ground stabilization scheme are based on the following criteria:

- Provide a safe and cost effective mining operation
- Minimize ground movement above the tunnel
- Mitigate the impact of tunnelling on the urban environment

### **2. GEOLOGICAL APPRECIATION**

As fully described in the geological reports, the Franciscan Sediments are largely shales, sandstones and greywackes. Weathering is commonly intense, along well defined joint and bedding planes usually steeply dipping. No large rock voids were detected although there are some secondary crystalline growths in very small cavities. Joints are typically of very small aperture but apparently of no great asperity. Clay coating and filling are common. The mass permeability of the



sequence appears to be low. Cores showed high recoveries but low RQD's. The material of the rock mass is typically competent and moderately hard when fresh. Core samples and outcrops reveal sharp edges to fissured samples.

All these observations suggest that this rock mass would have a "Moderately Low Groutability" (Deere, 1976) probably in the range of 25 - 50 kg/m of hole, even if supplementary cement grouting aids (such as superplasticizers, bentonite) or complementary microfine cements were used. Grout deposited in such fissures would typically be of high w/c ratio, and therefore of low strength properties, but would still be interfacing with (low strength) clay infill.

### 3. CONSIDERATION OF VARIOUS OPTIONS

Generically, given the decision that a mined or bored tunnel must be constructed as opposed to a cut and cover section, there are four possible groups of options for strata control and building settlement mitigation.

- (A) Conducting stabilization from the tunnel face(s) during advance.
- (B) Pretreat the whole alignment from the surface.
- (C) Underpin individually selected structures likely to be affected by the settlement trough.
- (D) Compensate for surface movements as the face advances, by grouting from discrete surface locations.

#### 3.1 *Stabilization from the Tunnel During Advance*

Ground reinforcement and/or treatment is conducted from the face, to enhance the stability of the ground around and in advance of the mining. Each face thus goes through a cycle of protect, excavate, support (figure 1).

- |            |   |
|------------|---|
| Advantages | <ul style="list-style-type: none"><li>- requires no surface work</li><li>- prevents settlements developing</li><li>- explores intimately geotechnical conditions in advance</li><li>- can proceed on multiple faces</li></ul> |
|------------|---|



- utilizes the self supporting capabilities (if any) of the rock mass
- can be adapted to different ground conditions throughout the tunnel

Disadvantages      -    needs careful sequencing with the tunnelling contractor

### 3.2    *Entire Pretreatment from the Surface*

This is a common principle which has been used frequently throughout the world, including recent soft ground tunnels in DC, Baltimore, Phoenix, Los Angeles, and San Francisco (e.g. Bruce and Pellegrino, 1995). Basically the soil is strengthened, and its permeability reduced, to permit safe, fast and continuous mining, without the need for compressed air or EPBS machines.

Advantages      -    work done in advance, no disruption to face working

- excellent in clays and silts (jet grouting) or coarser grained soils (permeation grouting)
- if executed at close centers gives clear geological picture

Disadvantages      -    needs complete surface access and presents other environmental concerns

- massive treatment can be very costly because of the low selectivity of the method
- rock masses not susceptible to significant "strengthening" by grouting

### 3.3    *Underpinning of Individual Structures*

Once the extent of the possible surface effects has been postulated, individual structures likely to be affected can be identified, and their susceptibility to differential settlements evaluated.

If deemed necessary, such structures can be protected in advance of tunneling by underpinning. In this instance, micropiles can be used in rock and grouting in soil.



- |               |   |
|---------------|---|
| Advantages    | - individual structures secured   |
| Disadvantages | <ul style="list-style-type: none"> <li>- would not prevent sudden collapses <u>between</u> supported buildings</li> <li>- need to place underpinning outside alignment of tunnel</li> <li>- access to foundations difficult</li> <li>- foundations may not be amenable to high bond stresses inherent in micropile system</li> <li>- movements may still occur</li> </ul> |

### 3.4 *Compensation for Surface Movements During Tunneling*

In this concept, the surface is allowed to move, but such settlements are compensated for by injecting grout into the subsoils, above and around the shield to "jack up" the ground. Compaction grouting has been traditionally used in the US (eg. DC, Baltimore, LA), while soil fracture grouting is being conducted on a huge scale in London, England.

- |               |  |
|---------------|--|
| Advantages    | - can give economic solutions in appropriate technical and logistical situations   |
| Disadvantages | <ul style="list-style-type: none"> <li>- only works in soils</li> <li>- would not stop sudden collapse</li> <li>- grout may enter excavation</li> <li>- despite high technological recent advances, success may be elusive.</li> </ul> |

For all these reasons, we consider that ground reinforcement/treatment from the face(s) is the best option.



#### 4. DEVELOPMENT OF CONCEPT FOR ROCK REINFORCEMENT / TREATMENT FROM TUNNEL FACE

With respect to the rock mass, it is believed that treatment by grouting alone would likely not produce a significant increase in mass strength, given the nature of the discontinuities. In addition, there is no recognized way of accurately measuring and controlling any benefit in strength, unlike the case with grouting for permeability reduction, where systematic water acceptance testing can be conducted.

Rather, we believe that if grouting of the rock mass be conducted at all for watertightness purposes, it is best done as a secondary operation to systematic in situ rock reinforcement.

The principles have long been known and have been employed, on a somewhat sporadic, "as locally needed" basis by U.S. tunnelers for decades under the names forepoling or spiling. This ground reinforcement method constitutes common tunnelling practice in Europe.

On large section tunnels, this method is commonly applied in conjunction with a two phase excavation: top heading and bench.

##### 4.1 *Top Heading*

The method consists of inserting a number of sub-horizontal reinforcing elements ahead and around the tunnel face to form an "umbrella-arch" over the succeeding tunnel section. The reinforcements (usually steel pipes) are driven into pre-drilled boreholes, and the annulus between the pipe and the ground is filled at low pressure with a cement grout. Specific drilling equipment is used both to drill the hole and drive the reinforcement (figure 2).

The steel pipe can also be equipped with sleeve ports and used as a grout pipe for high pressure grouting of the surrounding rock mass. As the grout penetrates and fills the rock fractures above and around the excavation, a reinforced arch is created. This grouting phase is usually critical in weak rock tunnelling, where a precise grouting program is implemented in order to avoid excessive decompression of the rock mass and facilitate the transfer of the loads on the temporary and final lining.

The length of one section of the reinforced arch is related to the rock characteristics and tunnel span, and is typically in the order of 12 - 15 m.



The extent of mining of each section will reflect confidence in the quality of the rock mass: in particularly troublesome or sensitive areas, the excavation can progress under "double cover" if wished. Typically, the superposition of two subsequent umbrella-arches is in the order of 3 m.

Each excavation advance is followed by the erection of steel ribs at appropriate intervals. Then the exposed surfaces are sprayed with shotcrete reinforced by fibers or wire mesh.

The arch excavation safely proceeds under "continuous beams" (the sub-horizontal pipes) on "multiple support" (the steel ribs). It is important that, in order to provide a continuous reinforcement around the tunnel and control deformations on the rock mass, the steel ribs have variable sections (figure 3). Special methods are available to pre-load the steel ribs and virtually avoid any decompression of the rock mass and therefore surface settlements. Evidently, the pipe section is chosen to withstand the loads transferred by the rock mass.

In case of extremely poor conditions of the rock mass, the stand-up time of the tunnel face can be increased by the installation of a temporary reinforcement. Sub-horizontal, sleeved fiberglass pipes can be easily driven from tunnel face and pressure grouted in place (figure 3).

#### 4.2 *Bench*

The subsequent phase of excavation is performed at a suitable distance from the top heading, in order to avoid interference between the two operations and concentrated stresses on the primary lining.

The excavation opens the lower half of the section, while the steel ribs are extended to the invert, in a manner similar to the top heading phase.

Given the properties of the rock mass, during benching support is likely required by the steel ribs erected in the first phase. This is important to maintain the carrying capacity and stress condition that the system acquired during the top heading excavation. Thus, the vertical thrust from the ribs can be transferred to micropiles founded at depths below final tunnel grade (figure 4).

In this event, steel ribs are bell shaped to easily accomodate the micropiles and spread the loads on a wider area.

Radial sub-horinzontal micropiles can also be adopted as "tie-backs" (active or passive) to minimize the horizontal convergence during the bench excavation.



The overall concept in these conditions is simply to ensure that the rock mass is nailed together, in such a way and at such close centers that no significantly large roof or wall blocks can detach, leading to a sudden collapse migrating to the surface (Bruce and Gallavresi, 1988). The spacing of the longitudinal (steel pipes) and transversal (steel ribs) elements is adjusted to form a supporting reticulate. Grouting, conducted from the same reinforcing members would seal any groutable fissures and reduce/eliminate water ingress.

This support system is used as a tunnel temporary lining. It can also be taken into account in the design of the permanent lining, thus permitting the use of a lighter lining.

## 5. TREATMENT OF SOFT GROUND

In essence, each of the various options listed above for rock mass treatment can be considered (Figures 5 and 6), although "forepoling" and rib support would have to be conducted with ground treatment techniques (jet grouting). The jet grouted umbrella-arch system has been successfully utilized to minimize settlements in a number of urban soft ground tunnels (McWilliam, 1991, Macchi, 1993)

The variable, but typically fine grained nature of the fills and natural ground would argue for jet grouting as opposed to permeation grouting.

Similarly, the historic nature of the area would argue for an extended pretreatment of the soil (i.e. trying to eliminate settlements at source), as opposed to compensating for later movements on delicate structures.

Micropiles could be used as underpinning for individual structures, but would be subject to the same types of disadvantages listed in 3.3 above.

## 6. GROUND REINFORCEMENT / TREATMENT SCHEMES

The following three ground reinforcement / treatment schemes are proposed:



#### A. Ground Reinforcement - Standard Section

This section consists of 19 sub-horizontal reinforcing elements, 40 feet long, installed from tunnel face on a conical surface with a 7 % slope (see figures 7, 8, 9). The excavation and installation of temporary lining proceed for 30 feet each section, under the coverage of the sub-horizontal piles. One subvertical micropile is installed on either side of each steel rib prior to bench excavation.

#### B. Ground Reinforcement - Sensitive Section

When mining below or close to sensitive structures, where very little settlements can be tolerated, the following changes are suggested to the Standard Section:

- increase the number of sub-horizontal elements on tunnel crown to 31 (see figures 10, 11).
- excavate only 20 feet per each section. This will provide a continuous double coverage of steel piles around the tunnel heading (see figure 12).
- install four additional micropiles per rib during the bench excavation, to minimize the risk of convergence on the lining (see figure 13).

#### C. Ground Treatment - Jet Grouting

Sub-horizontal jet grouting columns can be installed using the same drilling equipment described above for the forepoling. The jet grouting technique involves the use of very high pressure jets to destroy and simultaneously mix the soil in situ with a stabilizing grout. The application of this technique to the portion of the tunnel in soil, will create a concrete-like arch around the crown. Steel pipes will be then drilled inside the reinforced arch to provide additional load capacity across the steel ribs. The temporary lining shall be the same as in scheme A and B (steel ribs and reinforced shotcrete).

The three alternatives are summarized in table 1 below. The construction sequences relative to each alternative are described in the following chapter.



## 7. CONSTRUCTION SEQUENCE

### 7.1 *Ground Reinforcement*

#### 7.1.1 *Ground Reinforcement Installation on Tunnel Crown*

When starting the installation of sub-horizontal reinforcement on tunnel crown, the drill rig is advanced to tunnel face and positioned. The drill mast is aligned on the hole. The use of laser pointing devices is recommended in order to achieve the necessary accuracy in this phase.

The drilling sequence has to be designed in order to avoid interferences between adjacent holes. In general, two adjacent holes can not be drilled at the same time, unless one has been grouted.

As the drilling proceeds along the crown, the reinforcing members are inserted into the holes and grouted in place. The installation of a full "umbrella" consisting of 19-31 each 40-foot elements (including drilling, insertion and grouting) typically requires 2 to 4 8-hour shifts.

#### 7.1.2 *Top Heading Excavation*

Mining can start immediately after the withdrawal of the drill rig from tunnel face. The excavation span before the installation of double steel ribs and shotcrete is related to the rock mass quality and to the maximum tolerable settlement. Mining of the 30-foot tunnel length, including the installation of ribs, wire mesh, and shotcrete, should not take more than six 8-hour shifts.

It is anticipated that a minimum of two mining fronts are available and that the mining operation is performed on a three 8-hour shifts, five days a week. In this manner a minimum of 70 - 80 feet of tunnel can be reinforced, excavated, and lined each week.

#### 7.1.3 *Rib Foundation and Bench Reinforcement*

When the top heading excavation has been completed for a minimum of 150 feet, the installation of micropiles for rib foundation can



proceed. This span is deemed to be necessary to avoid interference between the two construction phases.

Micropiles are drilled inside the double ribs at different angles (see figures 7, 10). The steel members are fitted with valved grout ports to allow pressure grouting and increase the bonding between the reinforcement and the surrounding rock. This operation can be completed without disruption to the mining operation.

#### 7.1.4 *Bench Excavation*

The excavation of the bench proceeds when the crown is secured with micropiles. During the excavation, the steel ribs supporting the crown are extended to the invert along with the reinforced shotcrete. After the completion of the excavation and installation of a waterproofing membrane, the invert is poured.

The benching phase of Scheme B includes the installation of sub-horizontal micropiles. The excavation sequence proceeds as described above for Scheme A. When the excavation reaches a depth of 5 feet below the top heading floor, the micropiles are installed and grouted in place. The micropiles are installed at split spaces with the double ribs (see figure 13). A steel beam transfers the load to the ribs.

## 7.2 *Soil Treatment*

### 7.2.1 *Soil Treatment at Tunnel Crown*

The overall sequence for this operation is very similar to the one depicted for ground reinforcement in paragraph 7.1.1. The jet grouting columns are drilled and grouted from tunnel face using suitable drilling equipment. Once the jet grouting phase is completed on the top heading, sub-horizontal steel pipes are drilled inside the grouted zone to provide additional load carrying capacity to the system.



### 7.2.2 *Top Heading Excavation*

The mining of the top heading proceeds as in the rock reinforcement scheme.

### 7.2.3 *Rib Foundation and Bench Treatment*

The soil surrounding the bench is treated by jet grouting to provide a solid foundation to the steel ribs and lateral support during the benching phase. Jet grouting columns are installed at different angles (see figure 5) from the top heading excavation floor. This operation can be completed without disruption to the mining operation.

### 7.2.4 *Bench Excavation*

This excavation stage proceeds as described for Scheme A, and under the protection of the jet grouted soil.

## 8. SIDE DRIFTS OPTION

A two side-drift option has been considered for this project. Although we believe that this is likely not the most economically attractive option, the installation of a spiling canopy is hereby disussed.

After the excavation of the two side drifts has been completed (excavation phases 1 and 2), the installation of the spiling canopy can proceed. Approximately 15 sub-horizontal elements 40 feet long are installed in the same fashion described above for Schemes A and B (see figure 14, adapted from Haley & Aldrich). The excavation of sectors 3A and 3B can be extended for 30 feet, leaving 10 feet of overlapping with the successive canopy.

Before proceeding with phase 4 of the excavation, ribs foundations have to be secured with micropiles, drilled below the depth of the invert.



## 9. COSTING

A cost estimate has been performed for the ground reinforcement/treatment discussed above. The following data are to be considered as information base for lineal foot of excavated tunnel.

- Scheme A: 1,550 \$/lf
- Scheme B: 2,250 \$/lf
- Scheme C: 2,100 \$/lf
- Scheme D: 1,150 \$/lf

GP

Report No. S9612-03



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FIGURES



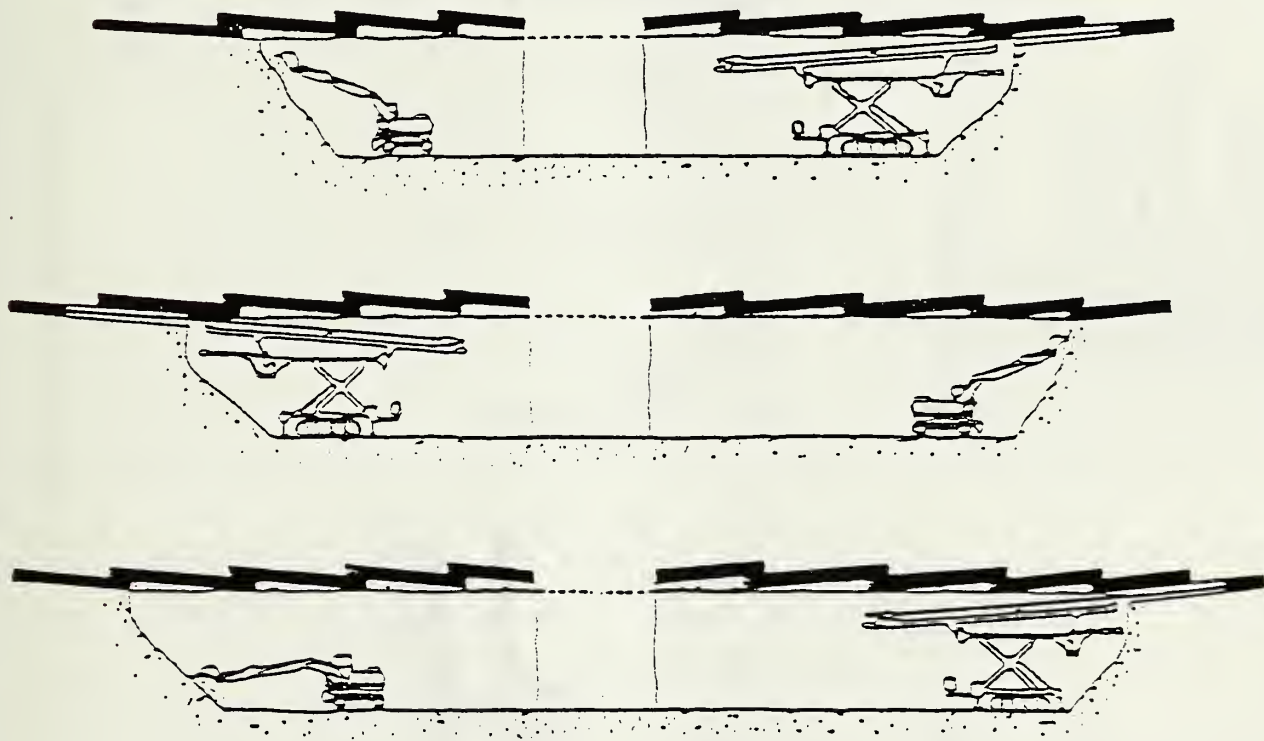
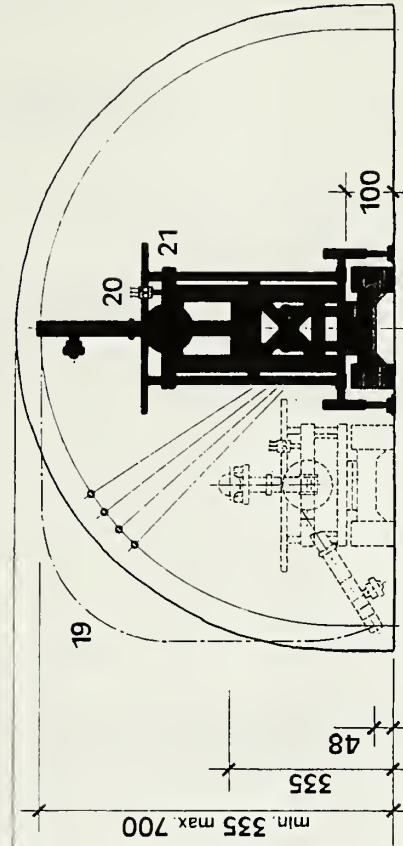
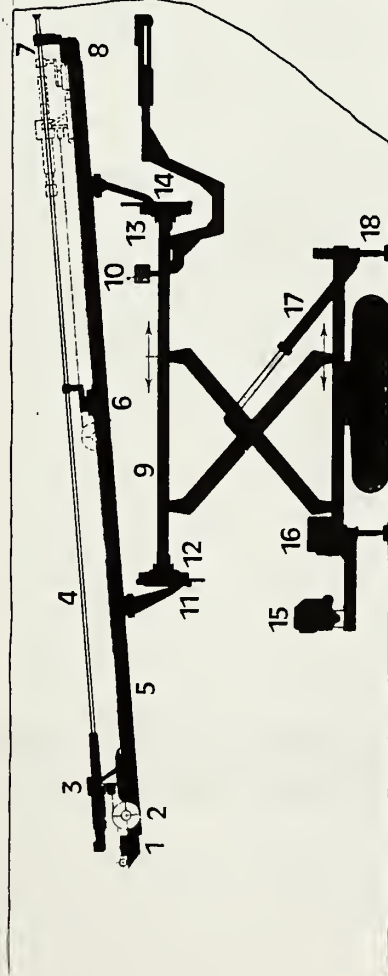
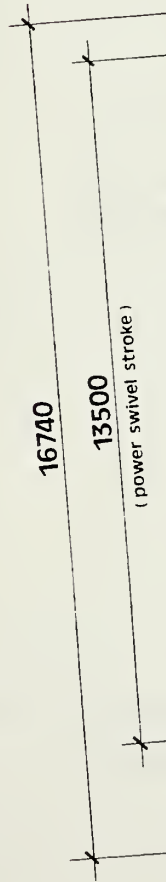


Figure 1 - Sequence of ground support installation and excavation stages on two headings.



# SR 510 · perforatrice · drilling rig



- |    |   |  |
|----|---|--|
| 1  | organo d'estrazione e controllo automatico estrazione | pull out winch and controlled extraction mechanism |
| 2  | avvolgitore per tubi flessibili                       | hydraulic hose take-up reel                        |
| 3  | testa di rotazione                                    | power swivel                                       |
| 4  | asta di perforazione                                  | drilling and jetting rod                           |
| 5  | mast  | mast   |
| 6  | controllo guida asta                                  | sliding rod guide                                  |
| 7  | morsa   | clamp  |
| 8  | organo di spinta                                      | pull down winch                                    |
| 9  | piattaforma di lavoro                                 | working platform                                   |
| 10 | pannello di comando                                   | control panel                                      |
| 11 | guida per inclinazione mast                           | inclination mast slide                             |
| 12 | ralla posteriore                                      | rear slewing ring                                  |

- |    |   |                                 |
|----|---|---------------------------------|
| 13 | ralla anteriore                             | front slewing ring              |
| 14 | guida per inclinazione mast                 | inclination mast slide          |
| 15 | centralina                                  | power pack                      |
| 16 | serbatoio                                   | hydraulic tank                  |
| 17 | cilindri sollevamento piattaforma di lavoro | working platform lifting device |
| 18 | cilindri stabilizzatori                     | stabilizer                      |
| 19 | limite dell'area perforabile dalla sonda    | border of allowed drilling area |
| 20 | pannello di comando                         | control panel                   |
| 21 | guida traslazione ralla rotazione mast      | slewing ring translation rail   |



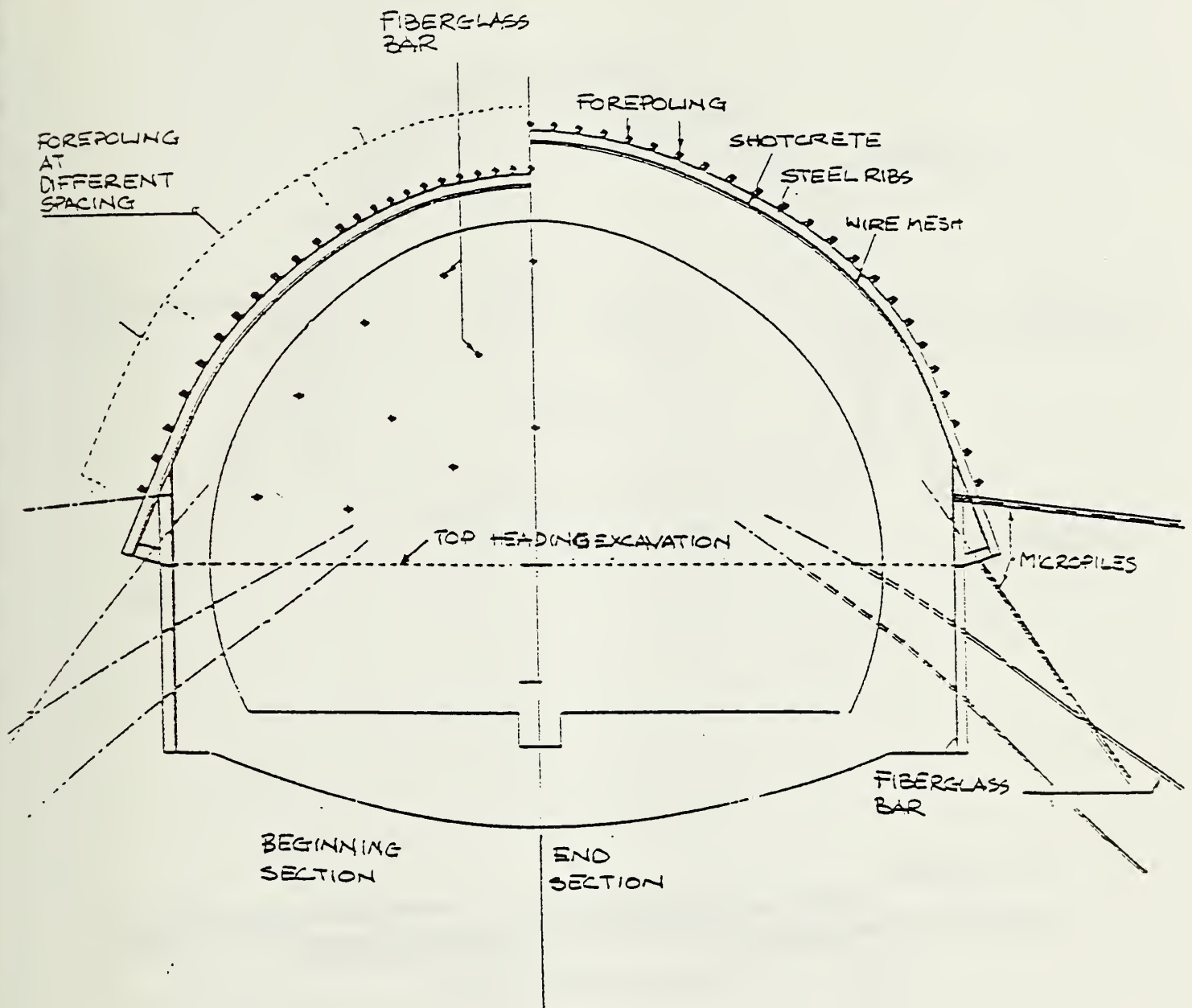


Figure 3 - Cross sections of a typical "umbrella-arch". Cross sections at the beginning and at the end of a reinforced segment.



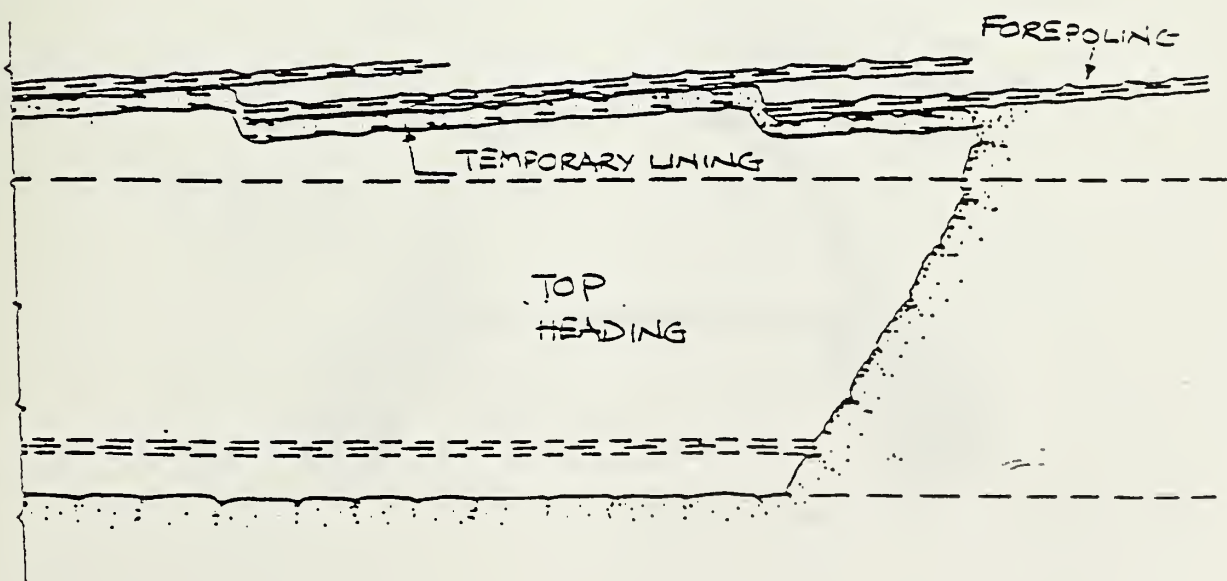
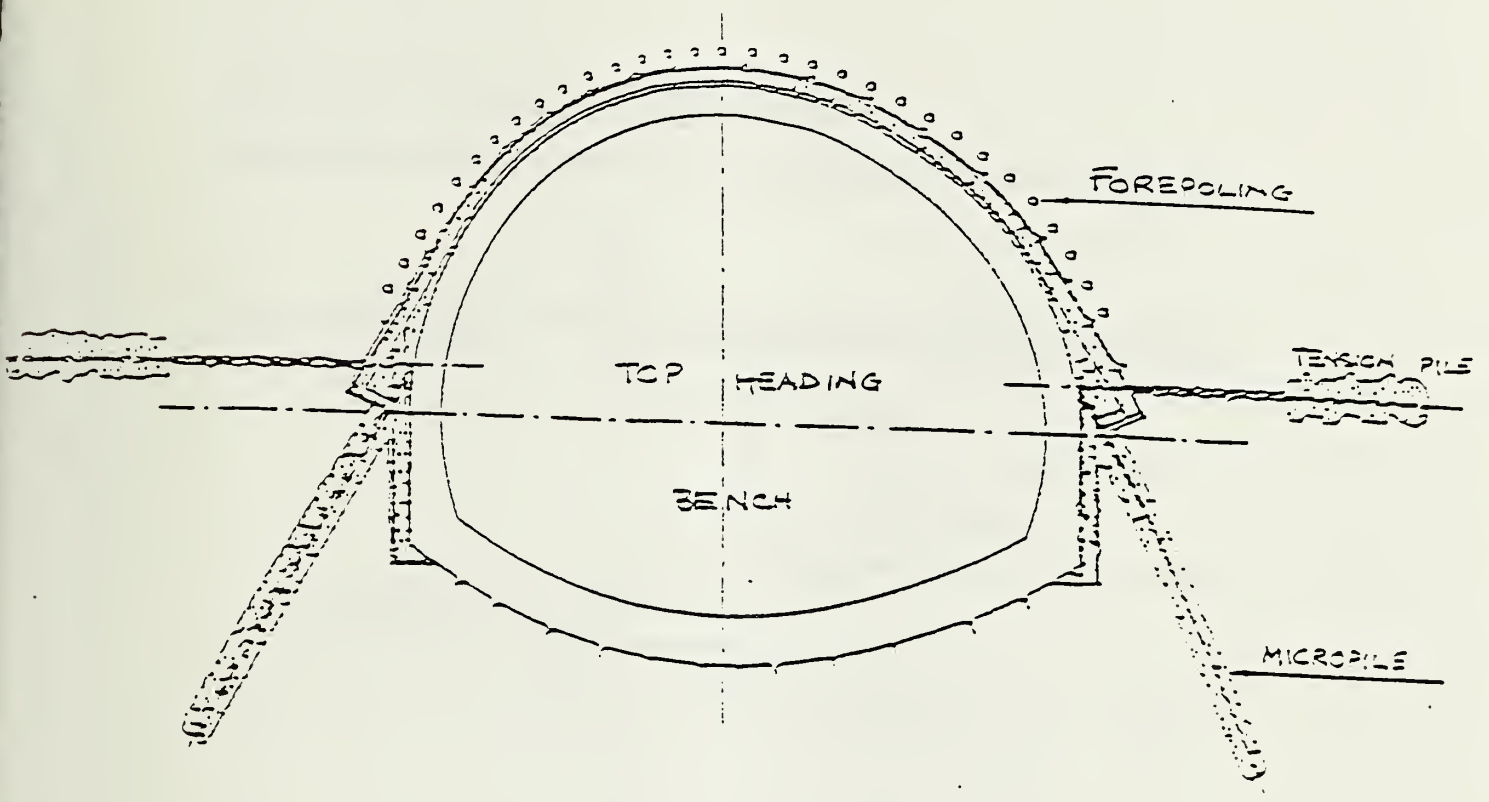
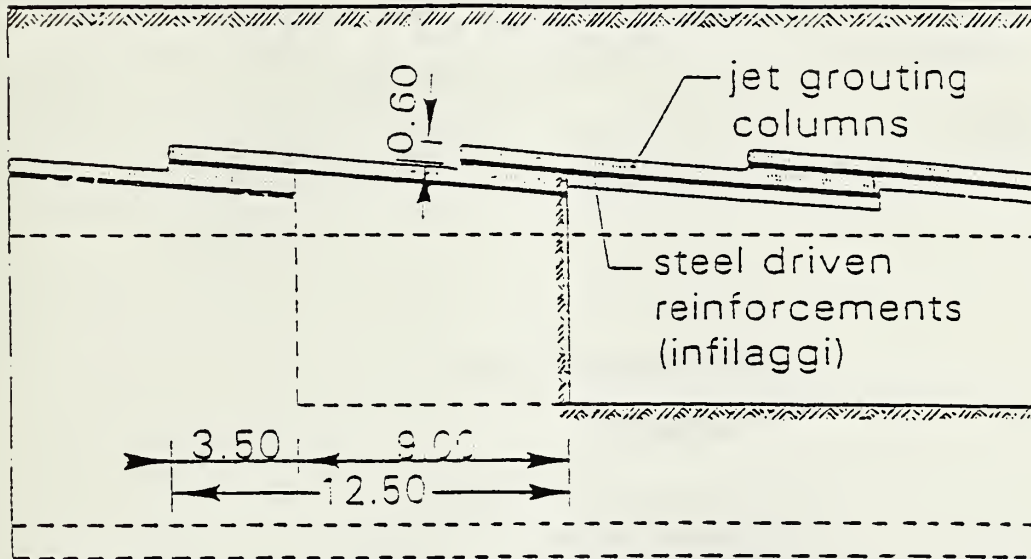


Figure 4 - Reinforced "umbrella-arch": longitudinal and cross sections. Rib supporting and "tension" micropiles are shown.



## LONGITUDINAL SECTION



## CROSS SECTION

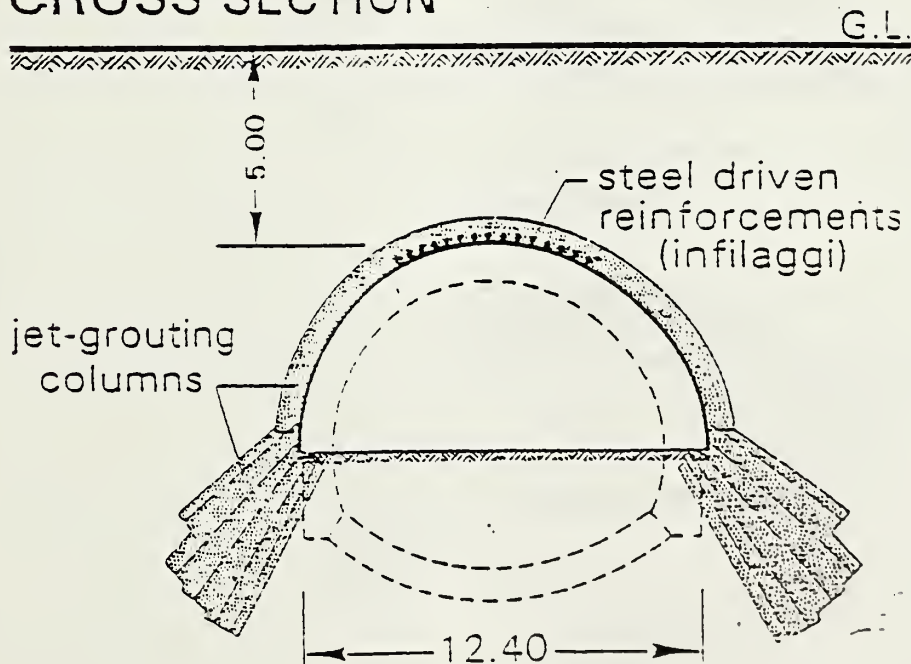
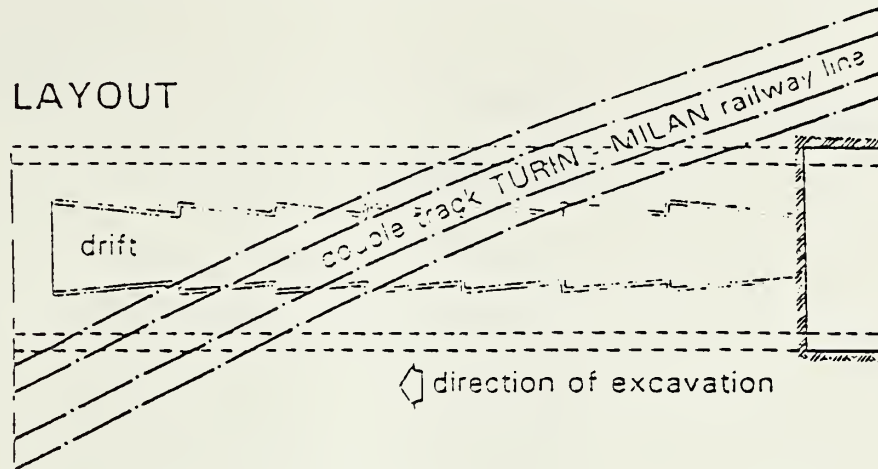


Figure 5 - Use of sub-horizontal reinforced jet grouting and sub-vertical jet grouting for soil improvement in shallow, soft ground tunnelling.

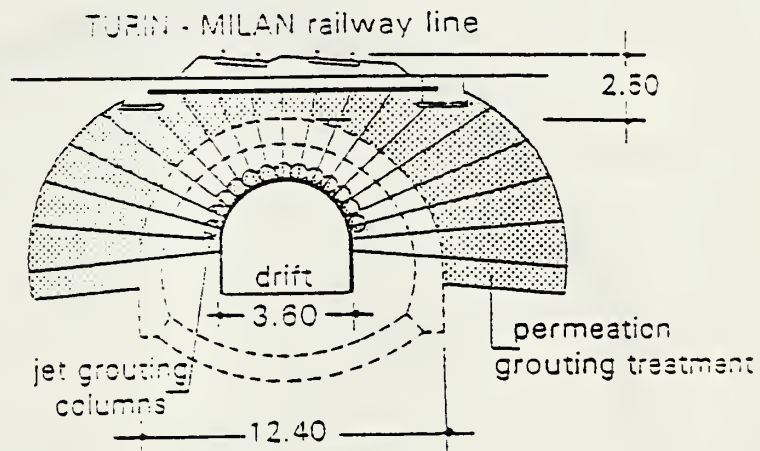


# LAYOUT



## CROSS SECTION

### 1st phase



### 2nd phase

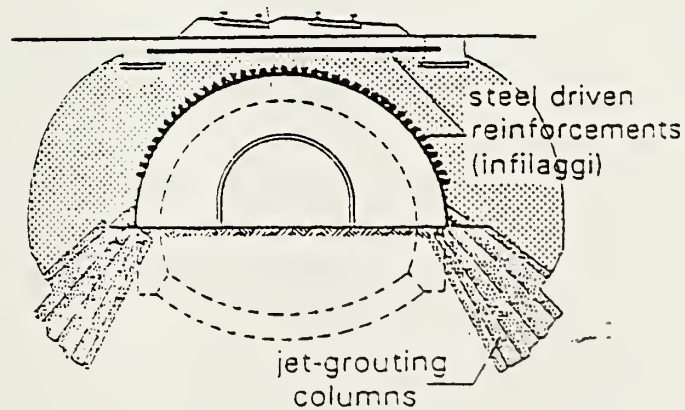
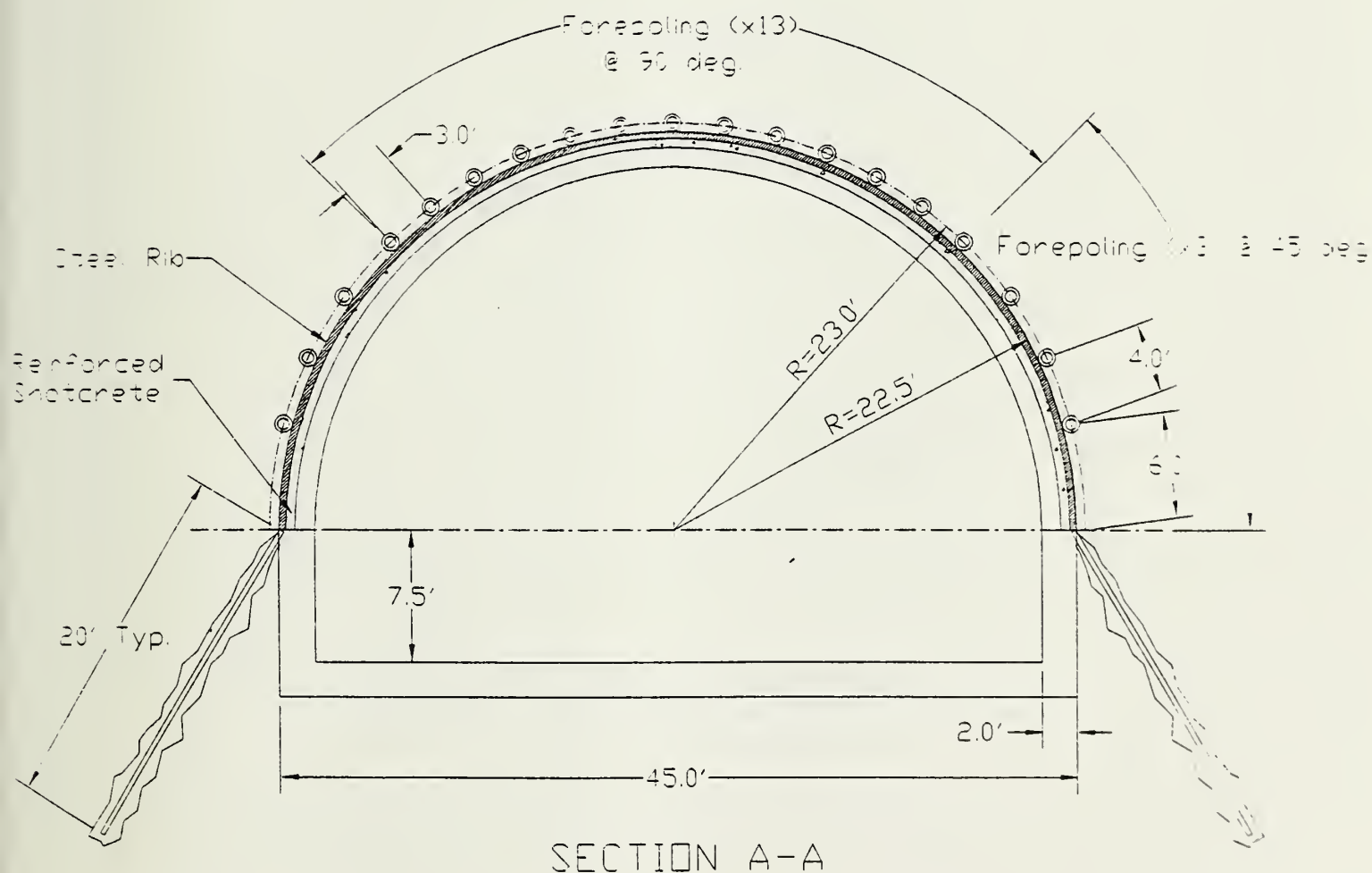


Figure 6 - An example of combined use of ground support and treatment in shallow tunnelling.



## SCHEME A - STANDARD SECTION

(Total Forepoling = 19)



SECTION A-A

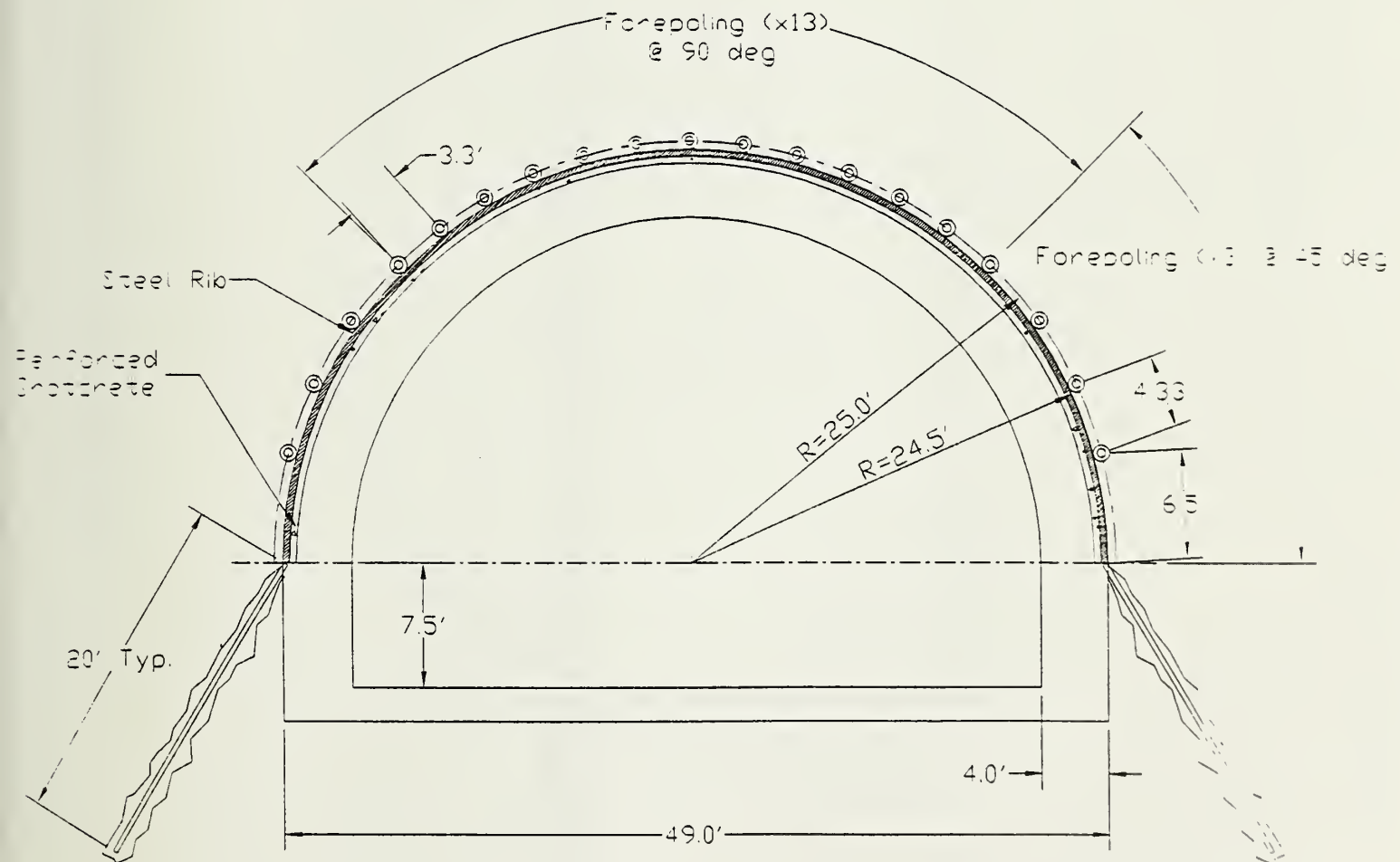
Note: All dimensions subject to final design

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## SCHEME A - STANDARD SECTION

(Total Forepoling = 19)



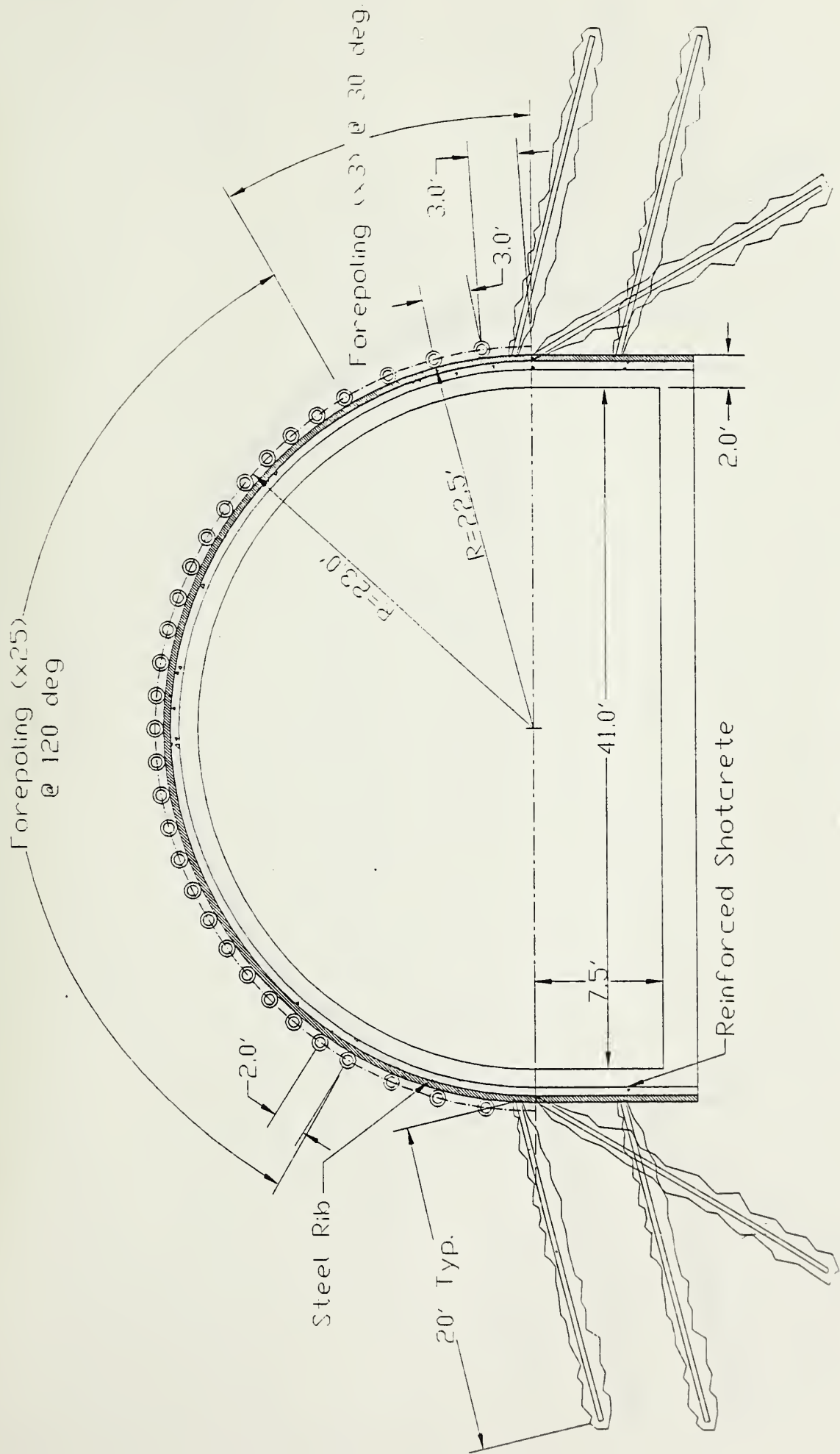
SECTION B-B







FIGURE 10  
 (Total Forepoling = 3D)

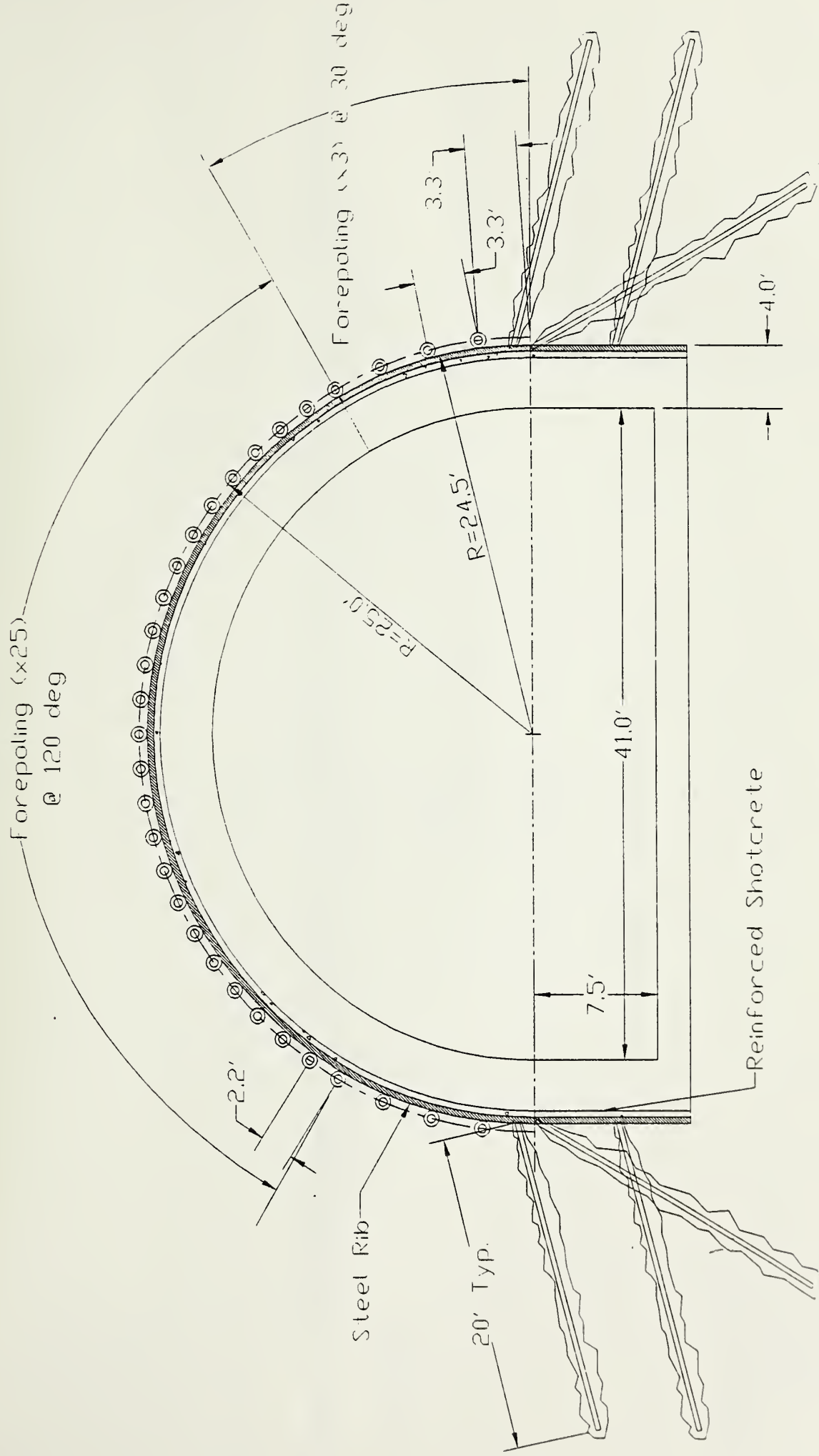


CUTLINE A-A



(Total Forepoling = 31)

FIGURE 11

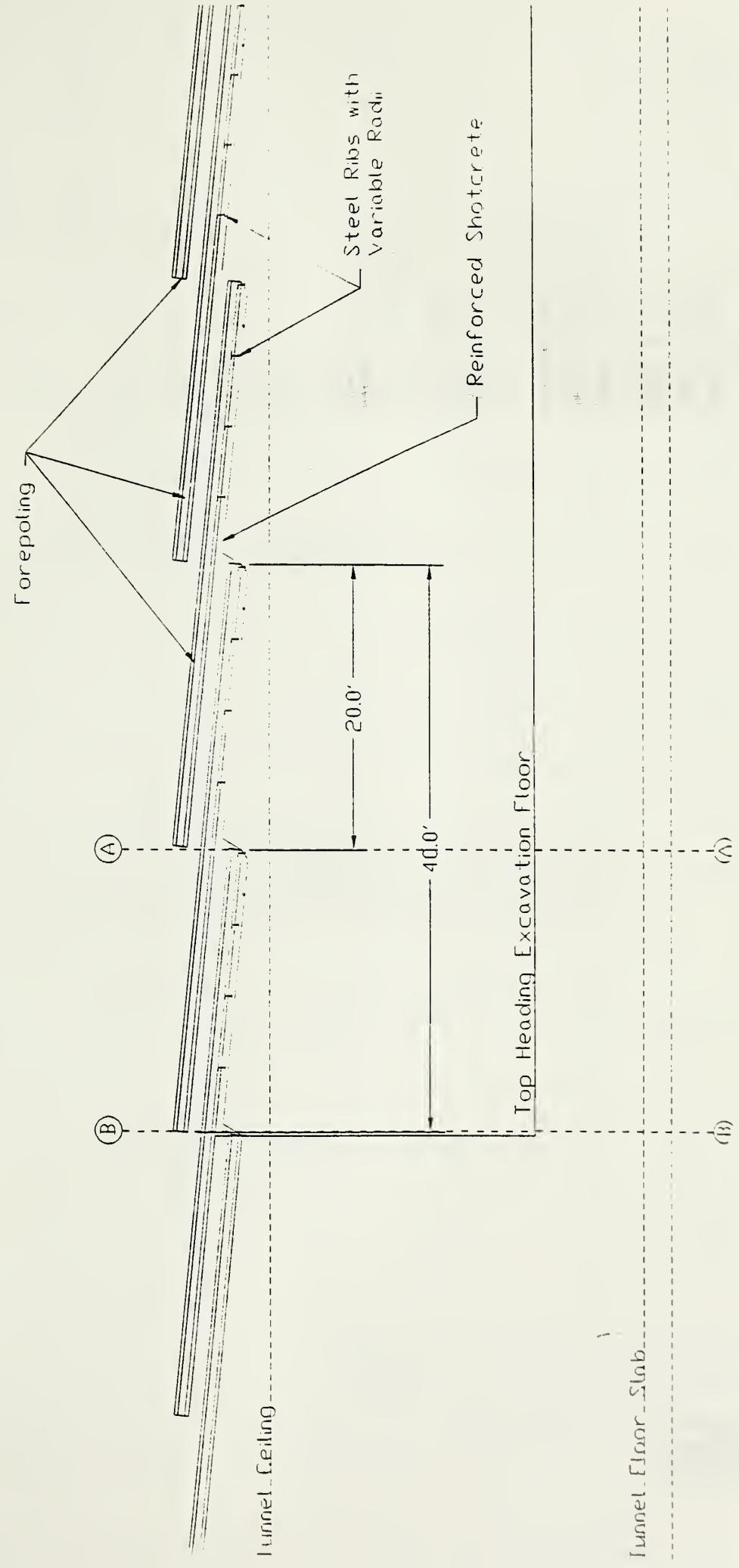


Note: All dimensions subject to final design.

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SCHEME B - SENSITIVE SECTION

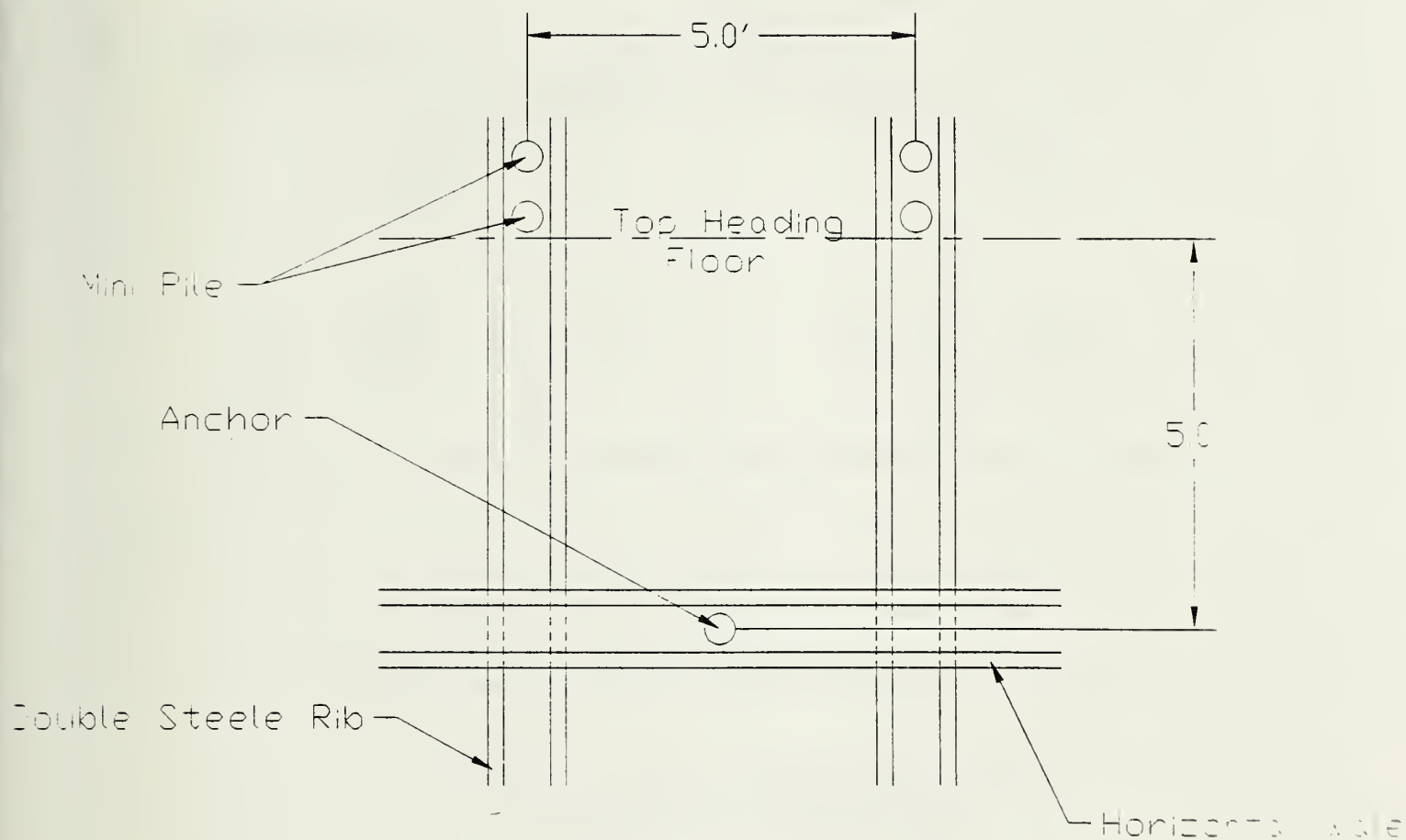


1' 1' 1' 1' 1'

Note: All dimensions subject to final design



## SCHEME B - SENSITIVE SECTION



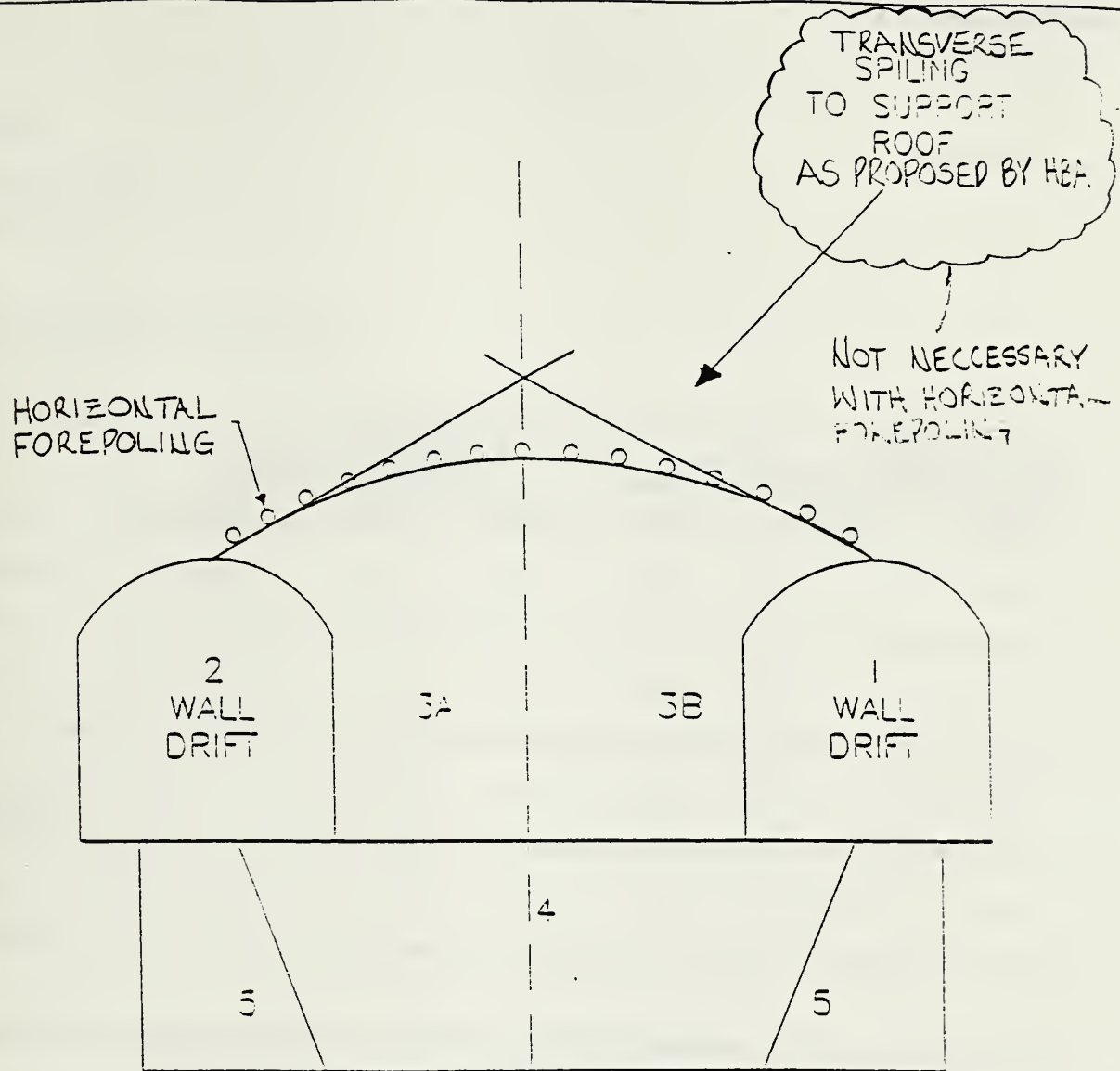
## SIDE WALL PROFILE

Note: All dimensions subject to final design.

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FIGURE 14



NOTE: NUMBERS REPRESENT THE  
SEQUENCE OF EXCAVATION

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CALTRAIN SR/ES  
TUNNEL CROSS SECTION  
PROJECT CITY AND STATE

SINGLE TUNNEL ALTERNATE

SCALE: NOT TO SCALE

APRIL 1998



**CALTRAIN**  
Downtown Extension  
San Francisco, CA

TABLE 1

Suggested Ground Reinforcement Schemes

	Top Heading				Bench	
	Forepoling/Jet Grouting			Excavation length	Micropiles/Jet Grouting	
	diam./thick.	each	length		each/rib	length
A. Standard Section	4" x 1/2"	19	40'	30'	2	20'
B. Sensitive Section	4.5" x 1/2"	31	40'	20'	6	20'
C. Soil Improvement (jet grouting)	24"	37	40'	30'	4	20'
D. Side Drifts (forepoling)	4.5" x 1/2"	15	40'	30'	N/A	N/A

Table 1: Suggested Ground Reinforcement / Treatment Schemes (all measures in feet, except as noted).



CASE HISTORIES



# Special Tunnelling Methods for Settlement Control: Infilaggi and Premilling

D.A. Bruce  
ARCON, Pittsburgh, Pennsylvania

F. Gallavresi  
Giovanni Rodio & Co., Milano, Italy

## SYNOPSIS

This paper provides an introduction to two tunneling methods specially developed to optimize settlement control. This is particularly relevant in urban environments.

The concept of neither method may be regarded as novel: infilaggi is a development of the principles of forepoling, whilst the premilling idea has been considered for some years. However, recent trends in the nature of the tunnel market, and major advances in the equipment and systems involved have fostered a rapid growth in Western Europe. A description is provided of the major points of each method, and case history data are cited to illustrate their excellent performance.

## 1. INTRODUCTION

When tunneling in urban environments often at relatively shallow depths and in variable ground conditions, the development of surface settlements is an attendant reality. Depending on the local circumstances, such settlements may be deemed too small to be of significance. Alternatively some form of settlement mitigation or correction may be necessary, for example grouting (Bruce, 1987) or insitu reinforcement (Bruce et al., 1987). Such methods, and others, will also aid the progress and safety of the tunneling contractor and his personnel (Mongilardi and Tornaghi, 1986).

On the other hand it may be more practical or economic to attack the problem at source--to isolate the impact of the excavation method from the surrounding ground, which would thereby retain its virgin status, and not affect overlying structures.

This paper describes two particular techniques, infilaggi and premilling, which have been developed to minimize ground movements above tunnels. Although entirely different in execution, they share many common features, principally:

- they are executed from the face in advance of excavation, thereby minimizing subsequent decompression.
- they are not new ideas--infilaggi has been known as forepoling or spiling in the U.S. for decades, whilst the premilling system has taken some time to refine to its current status; however,
- given current trends of shallow tunneling in urban areas both have a revived potential rapidly being realized in Western Europe in particular.
- recent developments in equipment technology have broadened that potential with respect to speed, reliability and cost effectiveness.

Each technique is described below, illustrated by reference to recent applications by Ing. Giovanni Rodio & Co., in Italy.

## 2. INFILAGGI

The use of forepoling or spiling has long been common as a supplemental support method in U.S. tunneling practice. For example, Clough (1981) described how the array of steel rebars

driven suprahorizontally around the crown forms an umbrella of reinforced ground above the subsequent excavation. However, within the last few years in Western Europe, the concept has been expanded upon, to the extent that horizontal insitu reinforcement or retention of this type (infilaggi) has become a primary tunnelling method. This has arisen for three main reasons:

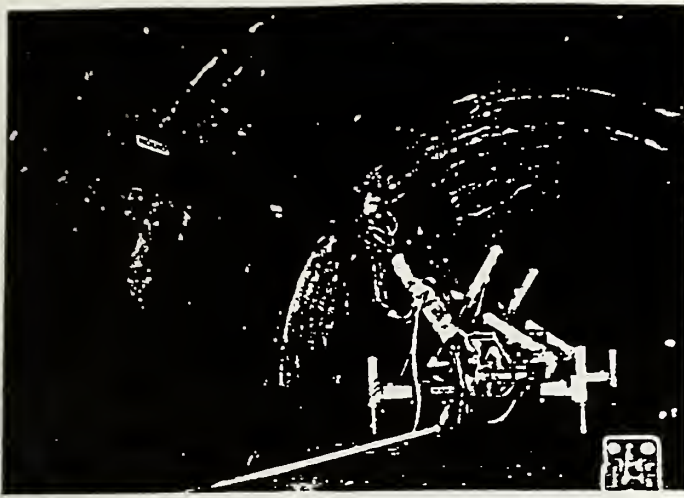
- Demand - there is a vibrant phase of tunnel construction in certain countries in Western Europe--especially Italy, France, Germany and Austria--for new or upgraded transport networks. Much of the construction is in ground which can be classified as soil or rock of inferior mechanical characteristics.
- Equipment - recent advances in specialized drilling equipment (e.g., RODIO SR500 [Photograph 1] and SR510 diesel hydraulic machines) can allow up to 20m of protection to be installed in one pass, with very high productivities and with minimal demands on the facilities or assistance of the excavation contractor.
- Contractual - the contractual atmosphere fosters innovation and the principles of risk sharing. Thus individual companies, or groups of companies are encouraged to evolve new concepts to improve cost effectiveness and productivity. Equally, these ideas may allow contractors to minimize initial capital expenditure, by virtue of using or modifying existing equipment.

The length of each pass of infilaggi varies according to the ground conditions and the balance between the various aspects of the tunnel construction processes necessary to ensure efficient utilization of all resources by all parties. In certain instances, the steel pipes forming the reinforcement may also be used for grouting the surrounding ground, and the use of sleeved ports for example permits a range of cements and chemicals to be used, depending on ground conditions and the support requirements.

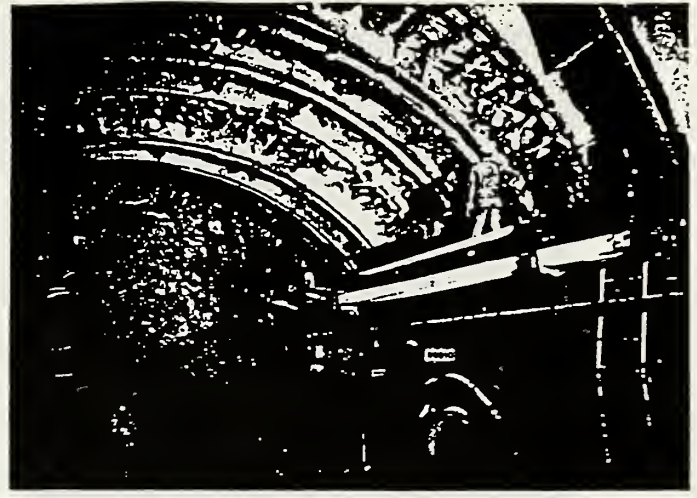
## CASE HISTORY: LIMINA TUNNEL MAMMOLA ITALY

The Limina motorway tunnel is located in the south of Italy, near Mammola, approximately 20 Km northeast of Reggio di Calabria. Typically, the 12m diameter tunnel has the profile of Figure 1, but at 250m intervals it opens out by 4m for a 40m

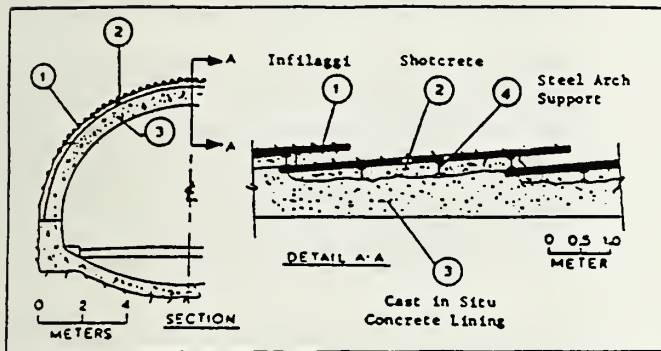




**Photograph 1** - SR500 diesel hydraulic tunnel drilling rig, installing 19m long, 140mm dia. infillaggi with sleeved ports to allow supplementary grouting of the morainic material. Up to 60 infillaggi per ring. For twin 12m dia. roadway tunnels. Dervio, Lake Como, Italy.



**Photograph 2** - Drilling for installation of infillaggi (seen stored on drilling rig). Earlier phases of infillaggi, ribbing and shotcreting seen above the rig. Limina Motorway Tunnel, Mammola, Italy.



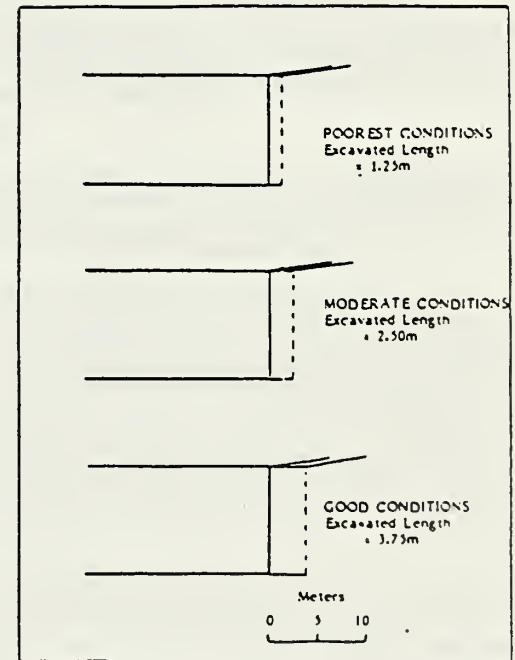
**Figure 1** - Tunnel cross section and details of in situ support (Infillaggi). Limina Motorway, Mammola, Italy.

length to provide passing points. The tunnel runs east-west and is 3100m long.

The bedrock is extremely variable hornblende-biotite diorite, shattered and weathered irregularly. It trends oblique to the axis of the tunnel and so each face can exhibit both "good" and "bad" rock. Fissure water is tapped by groups of four or five 100mm diameter drains 12m long drilled up from interim face locations as required.

In this instance, short infillaggi proved to be the safest and most cost effective solution, and a maximum installed length of 5.0m was determined (5.5m in enlargements). With the steel ribs being placed at 1.25m centers, this gave the option of excavating in steps from 1.25m (poorest ground) to 3.75m (best ground) long (Figure 2), depending on the rock conditions encountered. (To date the average has been 3.64m/drive.)

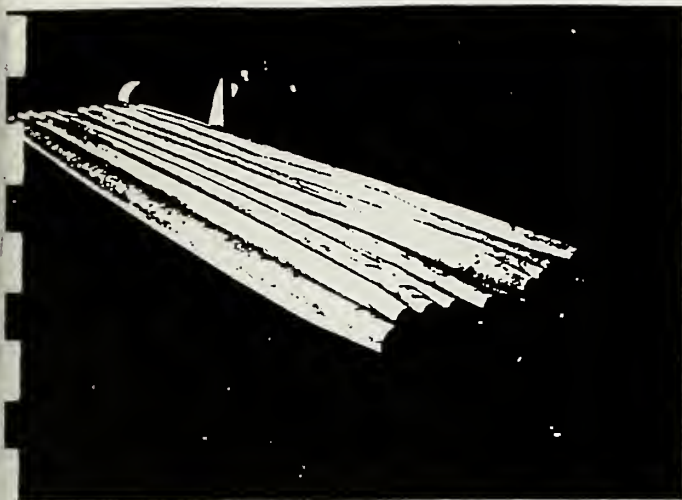
The long stroke SR500 or 510 rigs were not therefore required, and a purpose built truck-mounted diesel hydraulic rig



**Figure 2** - Simplified elevation, showing variation in excavation length under infillaggi, related to rock quality. Limina Motorway Tunnel, Mammola, Italy.

was used (Photograph 2). This drilled holes of 120mm diameter using foam flush (to minimize water flush requirements) through a hydraulic roto-percussive top hammer. The 45mm drill rods were "sleeved" to 90mm od. to ensure hole straightness.





**Photograph 3** - Infilaggi being assembled, showing fabric sleeve, sealed at top and bottom.

The holes in each umbrella were drilled at 300-400 mm centers (depending on rock quality) with an upwards inclination of 8°, but parallel to the tunnel axis. Drilling commenced in the poorest rock zones. In the normal section there were about 30 holes, with 40 in the enlarged sections.

Grouting the steel reinforcements into these holes required a novel solution, both to overcome the normal problem of grouting upwards, and to provide a high early strength performance. Each reinforcement consisted of a 76 mm o.d. steel pipe, sealed at the leading end. Rubber sleeves (manchettes) cover groups of four 6 mm diameter holes drilled in the pipe 1 m from either end. The entire length is encased in a continuous woven polypropylene geotextile jacket, closed against the steel at each end with steel clips and tape (**Photograph 3**). It is capable of unconfined expansion to 180 mm diameter.

The prefabricated reinforcement was placed in the borehole (**Photograph 4**) and a mechanical single packer introduced to just above the lower sleeve. A thick, neat cement grout was then injected through the packer to pass through the sleeve and so inflate the geotextile wrapping, forcing intimate contact with the borehole wall all along its length. In granular conditions (rock weathered to sand) injection was made with a silicate based grout, to permeate and consolidate the surrounding ground.

This system, permitting a very quick and clean installation and grouting operation, clearly solved the difficulties of "up grouting." In addition, the selected permeability of the geotextile material allowed cement grout water to be expressed during injection, thus leaving behind a grout of very low water-cement ratio, promoting fast set and very high early strength.

Following completion of each ring of holes, the drilling rig was withdrawn and excavation proceeded from the (lightly shotcreted) face. Steel arches were placed (as in **Photograph 4**), and the excavation then sprayed with up to 250 mm of shotcrete to ensure full contact of the structural elements and prevent local ravelling or seepage (**Figure 1**).

With this system, the average (and very satisfactory) rate of face advance was almost 3 m per day, and excellent cooperation between the infilaggi and tunneling contractors was recorded. Typically each infilaggi phase took eight hours



**Photograph 4** - Infilaggi being placed in predrilled holes, Limina Motorway Tunnel, Mammola, Italy.

followed by sixteen hours of excavation, arch placement and shotcreting. It has proved to be a particularly apposite solution given the flexibility it afforded in dealing with the major geological variations encountered.

### 3. PREMILL

Papers describing the successful development of "prédécoupage mécanique" (Premill) as a specialized tunneling technique in rock and soils have appeared sporadically in the French technical press over the last few years.

In essence, the system comprises a track-mounted frame, of shape corresponding to the tunnel extrados (**Figure 3**). Mounted on the frame and projecting out in front is a large band saw type milling machine about 3 m long (**Photograph 5**). This can be moved around the frame to cut a slot about 120-200 mm wide into the ground around the volume to be excavated. In competent rock this slot is left open, to optimize the subsequent blasting parameters and performance. In soft ground the slot is filled immediately with high strength, fast setting concrete, so forming an insitu arch to minimize decompression effects during subsequent excavation.

The system was evolved in response to the need to absolutely minimize construction-related effects in urban areas involving large diameter tunnels close to the surface under old and delicate structures. Under such conditions even the excellent performance afforded by the standard New Austrian Tunneling Method was not acceptable. The early examples for TGV and Metro construction in France have been followed by similar contracts for example in Lille (Belgium) and Roggiano (Italy), the last of which is described below.

In competent rock formations, of up to 2500 bar compressive strength, the premill is used only to provide a continuous slot around the volume to be excavated--usually with a drill and blast method (**Figure 4**). Premilling provides the following major benefits:





**Photograph 6** - Tunnel excavated under the pre-milled in situ arch, showing overlap of each cut, and placement of supporting steel ribs.

In comparison with the NATM, there are certain similarities, notably the overall concept of the support, and the common construction elements such as shotcrete, bolts, and arches. However, the major dissimilarity is that with premilling the primary lining is placed up to 3m ahead of the face before excavation, whereas in NATM the lining follows 1 or 2m behind the excavated face. This greatly impacts the generation and scale of tunnel deformations, and so the effect on overlying structures (Figure 5). Goër (1982) described a monitored case history of the relative performance of the two methods in the same material--Argenteuil marl (Figure 6). Typical properties of this material were listed as:

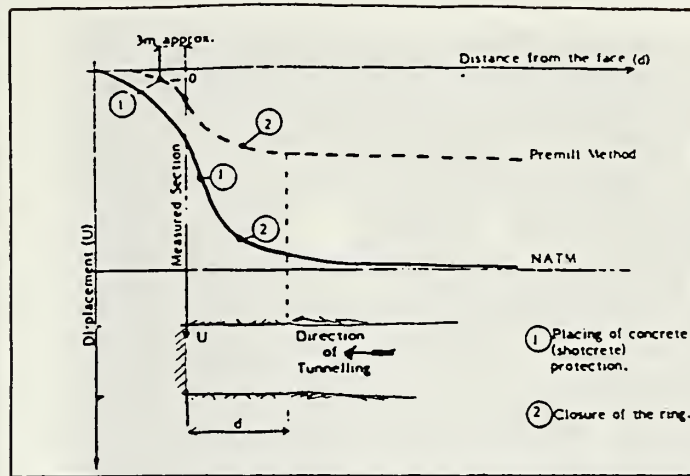
Density = 2  
Effective  $\phi$  = 20°  
Effective cohesion = 0.5 - 1.0 bar  
Undrained cohesion = 1.5 bar  
Deformation modulus = 500 bar

Three times less settlement was achieved in the tunnel protected by premilling.

Most of the earlier applications have been carried out with the conservative "divided section" profile. However, excellent results with the "full section" profile (i.e., cutting a 270° arc, and excavation in one pass) in a shallow circular collector tunnel 3.50m diameter (Département de la Seine-Saint-Denis, France), in very difficult ground, encouraged its use in Lot 7 of the Lille Metro, Belgium. As evident in Figure 7 the performance of the full section profile was superior, with surface settlements no more than 1mm. Prefabricated base slabs were connected structurally to the premill cover by shotcrete, and the steel ribs then placed, bearing on the slabs. This system also proved faster than the divided section approach.

#### CASE HISTORY: RAILWAY TUNNELS FROM MONGRASSANO TO SAN MARCO ARGENTANO NEAR ROGLIANO ITALY

During 1986 and 1987, three tunnels totalling 2000m in length were formed by premilling. The soil was generally lightly indurated and variable sediments, typified by waterbearing silty claystones with fine sand lenses. The profile of the tunnel is shown in Figure 8, the 140mm wide slot extending 21m around the shape and 3.5m beyond the face. Each cover overlapped the



**Figure 5** - Qualitative comparison of development of tunnel movements with Premill and NATM. (LeGoër, 1982).

earlier by 500mm, enabling the excavation to proceed full section in 3m drives, at an average rate of one advance per 24 hours, viz:

- Premill and shotcrete 8 hours
- Shotcrete stiffening period 6-8 hours
- Excavation and placing ribs 8-10 hours

The three faces were advanced simultaneously and laser guides used to ensure precise tunnel orientation.

The special dry shotcrete mix designed for this contract also incorporated steel fiber (about 30Kg/m<sup>3</sup>) to aid performance, and the mix reached 100 bar at 8 hours. Segments of 2-4m length were successively cut and filled.

During premilling in particularly poor conditions, the face (previously lightly shotcreted) was temporarily supported by simple mechanical props. Under these conditions, the spacing of the steel arches was halved. The final lining followed not more than 50m from the face.

#### **4. SUMMARY AND CONCLUSIONS**

The potential for the cost-effective use of infilaggi for soil and poor rock tunnelling has been greatly expanded by recent developments in drilling and grouting technologies. It is now realistic to anticipate single drives of over 15m, fully protected by the insitu reinforcement, and with or without complementary ground injection, using the new generation of purpose built drilling rigs.

In cases where surface settlements must be absolutely minimized then the premilling technique has been proved an excellent method, markedly superior in performance to the standard NATM. It can be carried out in both rock and soil.

The successful application of each technique, however, demands close and harmonious cooperation between the support specialist and the excavation/lining contractor. Once such links are forged in the U.S., the demands of the urban shallow tunnelling market will be more efficiently served.

#### ACKNOWLEDGEMENTS

The authors have pleasure in acknowledging the assistance of their colleagues at NiCON Corporation, Pittsburgh, PA, and at Ing. Giovanni Rodio & Co, Milano, Italy.



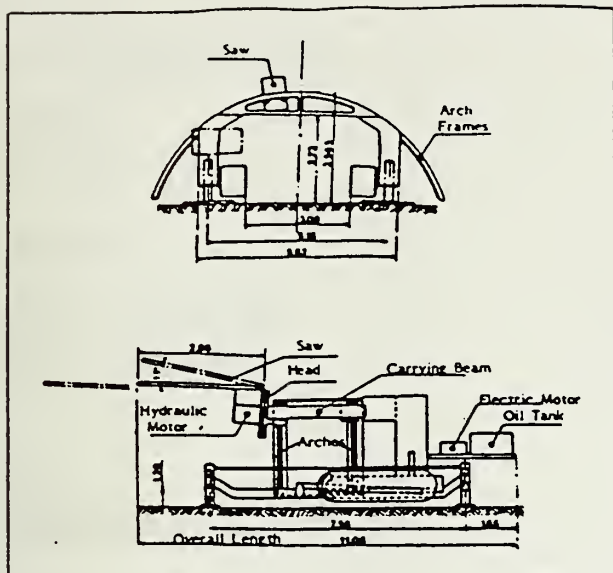


Figure 3 - General layout of typical premilling machine (Hydraulic arm for shotcreting not shown).

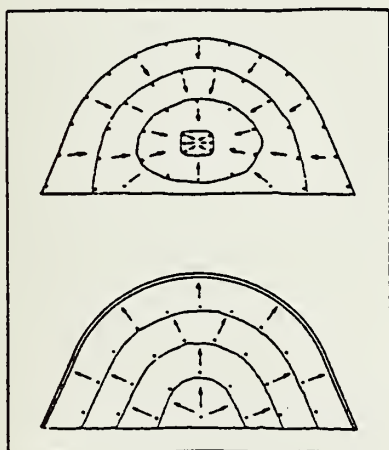


Figure 4 - Round blasting concepts - classical (upper) and with premilled slot (lower).

- less explosives (and blast holes) are required, rendering the entire blasting operation safer, faster and environmentally more acceptable.

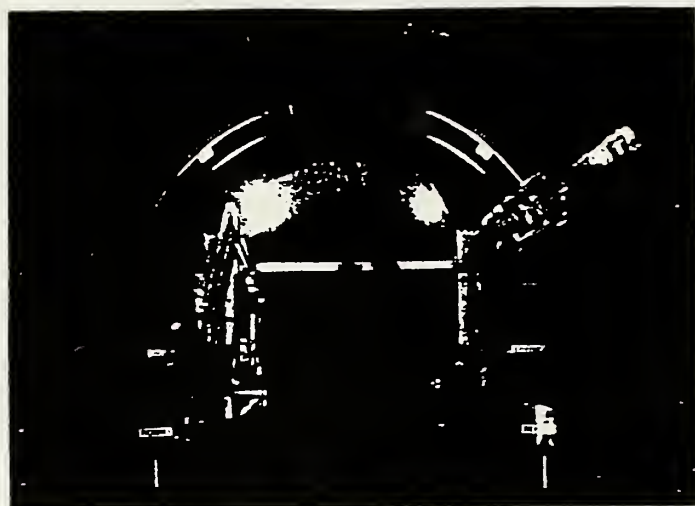
- no fissuring or decompression occurs in the surrounding rock mass, thus preserving its virgin properties, and so reducing the demand for subsequent reinforcement, e.g., with bolts.

- there is no overbreak, and therefore associated cost savings in time, effort and materials.

- the smooth profile makes the placing and performance of arches more efficient.

- less contact or consolidation grouting is needed behind the final tunnel lining.

- following blasting there is a greatly reduced danger from rock falls due to chimneys of fractured ground developing above the excavation.



Photograph 5 - Premilling machine, seen from the front and showing premilling saw (upper right) and hydraulic arm (left) for shotcreting operations.

- the magnitude of vibrations transmitted upwards towards nearby surface structures is greatly attenuated.

Developments of the technique continue, for example, in special diamond tools, high pressure water jetting, and increased cutting power, to permit its use in harder rock formations, faster, and with increased safety.

In soft ground, as noted above, the major difference is that the cut slot is filled with a special concrete mix as early as possible. The advantages are as identified above for the rock premill, although the prime target is the elimination of surface settlements induced by the tunnelling.

Each cover, up to 3.5m long, depending on the soil, is inclined slightly outwards and overlaps the preceding one by 300 to 500mm (Photograph 6). The cone is cut in discrete segments so that the concrete can be placed in each segment, as early as possible and without having to wait for the whole arch profile to be first completed. Cutting times for a typical 3m long segment may be as low as one minute.

The concrete may be placed by dry or wet shotcrete methods. A typical mix reported by Bougard et al (1979) comprises, per cubic meter of mix:

Cement	450Kg
Sand	560Kg
Fine gravel	650Kg
Coarse gravel	650Kg
Accelerator (Sigunite)	27Kg
Water, as appropriate, typically w/c - 0.25-0.30	

This gives a strength of up to 100 bar at eight hours. Replacement of the normal cement with 350Kg of cement fondu gave the minimum target strength of 80 bars in 4-5 hours. In addition, spraying the mix into the premilled slot ensures that none of the fine aggregate is lost, as is the case in conventional NATM applications of shotcrete on open faces. The concrete in place is, therefore, of superior quality, further enhancing the performance of the system.



Alessandro Macchi

IL NODO FERROVIARIO  
DI TORINO  
STATO DELL'ARTE

THE TURIN  
RAILWAY JUNCTION  
STATE-OF-THE-ART

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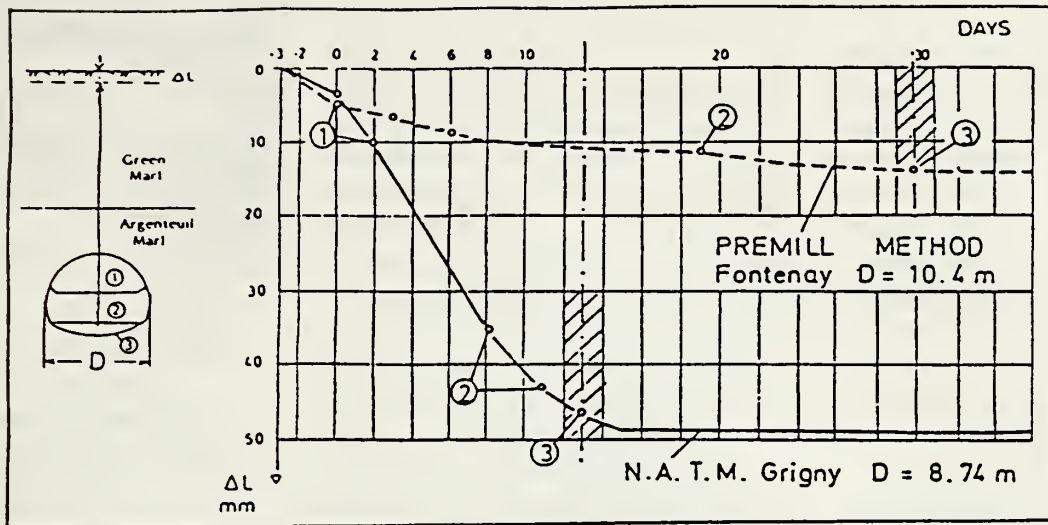


Figure 6 - Comparison of settlements generated with time in identical geological conditions, by Premill and NATM. (LeGoër, 1982).

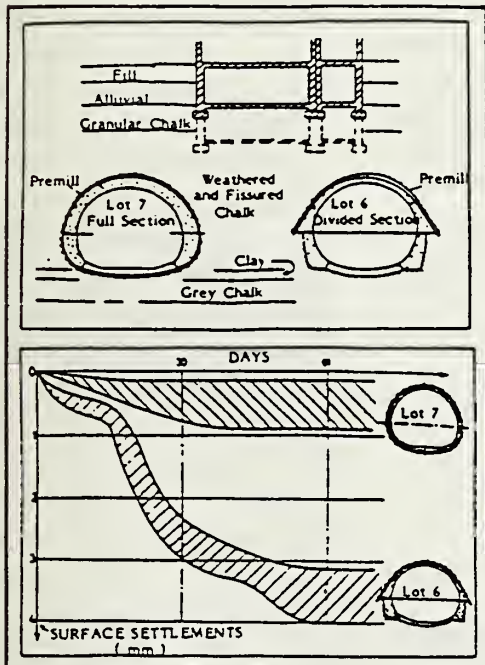


Figure 7 - Comparison of settlements generated by Premilling in Divided Section and Full Section, Lille, Belgium (LeGoër, 1982).

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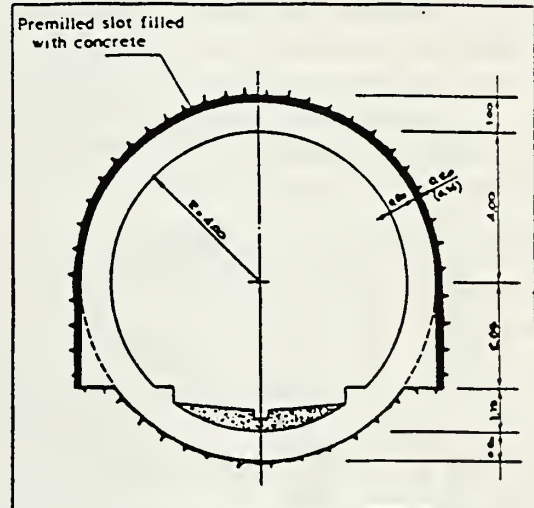


Figure 8 - Geometry of Roggiano Railway Tunnel, Italy, showing premilled protective arch.



## Alessandro Macchi Il nodo ferroviario di Torino Stato dell'arte

Il 16 gennaio '91 il Sindaco di Torino e l'Amministratore Straordinario dell'Ente Ferrovie dello Stato firmavano il protocollo d'intesa per il finanziamento congiunto del Nodo di Torino, un'opera di prerogative tali da configurarsi come la più importante per la città negli ultimi cinquanta anni.

Dopo decenni di completa stasi urbanistica, durante i quali né metropolitana né parcheggi sotterranei erano mai riusciti a decollare, finalmente, pur in un momento particolarmente difficile e complesso per l'economia nazionale e della città, Torino vede ora aprirsi una schiarita con i lavori del Nodo ferroviario.

La costruzione del Nodo assume infatti ruolo importante e decisivo per il futuro assetto urbanistico della grande città piemontese con la creazione della cosiddetta "Spina centrale", resa possibile dallo sprofondamento in sotterraneo del lungo attraversamento ferroviario urbano che da sempre ha diviso Torino, in trincea e a raso, longitudinalmente e parallelamente al Po.

### 1. Gli scopi dell'opera

Il progetto di sistemazione generale del Nodo di Torino, Fig. 1 consiste nella costruzione di un insieme di collegamenti che consentono di separare funzionalmente la rete ferroviaria per la media e lunga distanza dalla rete ferroviaria per il trasporto locale. Questo allo scopo di potere disporre di un'offerta di trasporto che sia più mirata alla domanda per soddisfare le richieste di ogni particolare clientela. Fino ad oggi le Ferrovie hanno, in Torino e quasi in tutto il resto d'Italia, una sola rete nella quale corrono sia i treni merci, sia i treni locali, sia quelli a lunga distanza. Ciascuno di essi viaggia con la sua velocità ed ha fermate in diverse località cosicché avviene che i treni più lenti interferiscono con quelli più veloci con la conseguenza che la quantità di treni ed il loro orario è più rispondente ai bisogni della circolazione che a quelli dei viaggiatori.

Il modo migliore per superare questi inconvenienti è riuscire a separare funzionalmente i due flussi di traffico, cioè quello delle medie e lunghe distanze da quello delle brevi distanze, in maniera tale da avere su ogni linea solo treni con caratteristiche omogenee.

Nel caso del Nodo di Torino questa separazione funzionale tra le reti è realizzata tramite la ristrutturazione della linea esistente, la costruzione della nuova linea costituente il quadruplicamento per Milano e la costruzione della nuova linea detta "passante".

Il passante attraversa non soltanto la città di Torino ma l'intero comprensorio ferroviario da sud verso nord, cioè dalla stazione di Trofarello fino alla stazione di Chivasso collegando fra di loro tutte le ferrovie di carattere locale che convergono su Torino, cioè le ferrovie che provengono dalla provincia di Cuneo, da quelle di Asti ed Alessandria, da Pinerolo, da Chieri per quanto riguarda la direzione sud, da Ceres e Caselle e dal Canavese per quanto riguarda la direzione nord e dalla Valle d'Aosta, dal Casalese e anche dalle province di Vercelli e Novara per quanto riguarda la direzione est. Tutte queste ferrovie confluyendo su questa nuova linea costituiscono una rete ferroviaria regionale la quale funzionalmente può essere esercita in maniera separata rispetto al resto.

Di conseguenza la rete esistente potenziata con il quadruplicamento per Milano potrà essere dedicata

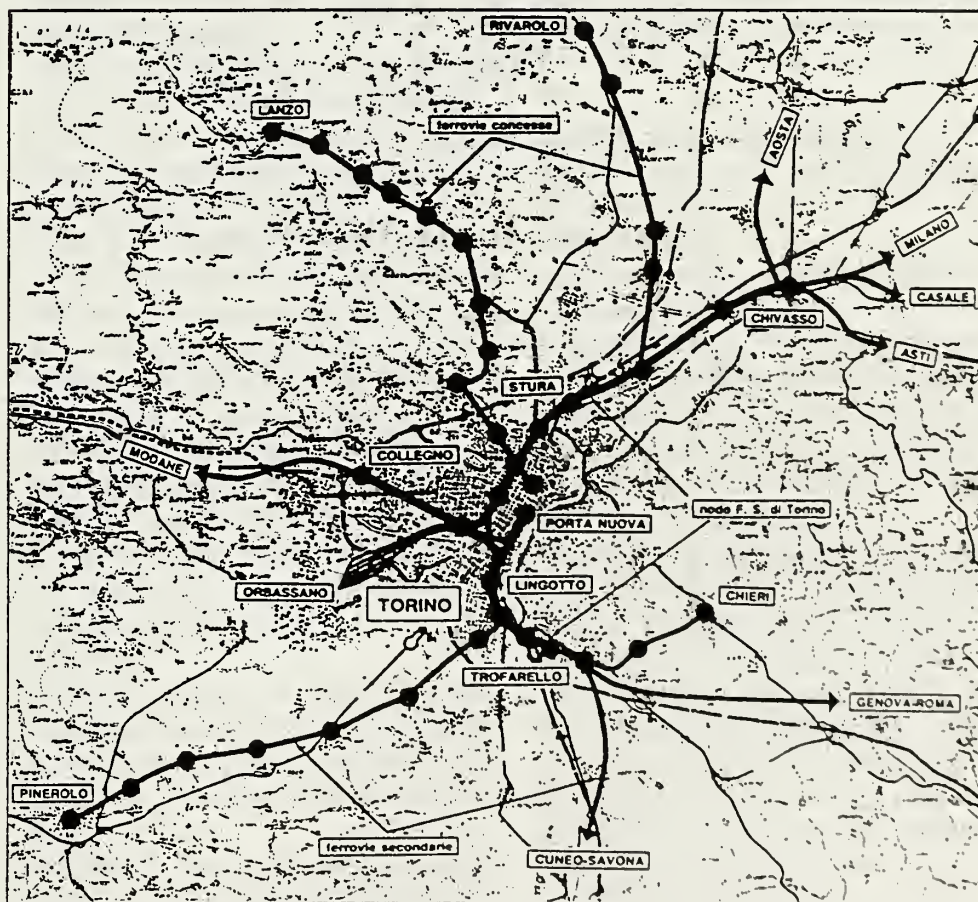


Fig. 1 - Il Nodo di Torino e le sue potenzialità  
The railway junction of Turin. A multiplier of potentiality



## 1. Objectives

The general location plan of Turin Railway Junction, Fig. 1, entails the construction of a number of connections to functionally separate medium- and long-distance railway lines from slow lines. The result of this is expected to be a transport supply more targeted on demand, able to meet the requirements of each specific customer. Up to now Italian Railways have had one single network for goods trains, slow trains and fast long-distance trains both in Turin and in the rest of Italy. Each train travels at its own speed and stops in different places, and as a consequence slower trains interfere with faster ones. The end result of this system is that the number of trains and their timetable is more in line with traffic rather than with passenger needs.

The solution to eliminate this drawback is a functional separation of the two different traffic flows, the medium and long distance flow from the short distance one, so that each line has specific trains with homogeneous characteristics.

In the case in point, the functional network separation of the Turin Railway Junction will occur through a restructuring of the existing line, the construction of a new one, that is the quadruplication of the Turin-Milan line, and the construction of the new so-called "connection through line".

The connection through line will not just cross the city of Turin but the whole railway district from South to North, that is from the Trofarello Station to that of Chivasso. It would therefore link all branch-lines converging on Turin, those from the districts of Cuneo, Asti, Alessandria, Pinerolo, Chieri southwards, and from Ceres, Caselle and Canavese northwards, and those from Valle D'Aosta, Casalese, and the Vercelli and Novara districts eastwards. All these railway lines would converge on the new line and become a slow local network functionally separated from the remaining routes.

As a consequence the existing network enhanced by the quadruplication of the Turin-Milan line will be devoted to medium-long distance traffic only and will connect Turin to the new high-speed line.

The Junction consists of a number of railway and civil works:

The current plan that was first drawn up in the Eighties and then supplemented by more specific railway and urban works laid down in the preliminary draft of the new Town Plan is now at the approval stage.

The setting up of these new railway lines is considered by the Turin Municipality as an opportunity to enhance the urban environment through the rail-

ways themselves. For this reason the Turin Municipality asked and obtained from Italian Railways the underground construction of the new lines, the "covering" of existing ones and the construction of a large avenue on them. The new railway works would thus be utilised to improve traffic, to link parts of the city previously separated by the railways, and to enhance all surrounding areas through a city redesign.

This made the construction of this railway work extremely complicated and challenging for designers who had to conciliate design difficulties with railway operation constraints and with the need of minimising inconvenience to the city.

Here is the Railway Junction construction progress: The Trofarello - Torino Lingotto route section has been operating for several years now. A first section of the Lingotto - To-Stura of two and a half kilometres astride the Stura river was started in 1986, all open, then works were stopped in 1989 in the framework of a priority review by Italian Railways and resumed in 1992; for the Lingotto - To Porta Susa functional route section the building site became operational in June.

The construction was entrusted to Recchi S.p.A. as parent company mandatory operating in collaboration with C.C.P.L., C.I.S. and FIAT Engineering.

## 2. Main structural features

They are characterised by two main configurations: (Fig. 2 and Fig. 3)

- natural tunnel
- artificial tunnel both for new lines and for the restructuring of existing ones

It should be noted that both railway and urban works are heavily influenced by railway operation and by existing buildings. Old and new structures geometrically interfere with each other, hence the need for highly complex interventions affecting vital underground and off-ground utilities.

From Lingotto, the starting point of the Route section, for about 1850 meters the connection through line runs through a cutting next to the existing Turin-Genoa line, then it runs deeper underground in an artificial tunnel. At Corso Bramante, one of the main avenues crossing the city from East to West, the line starts exactly beneath the extreme street span underpassing it with the rail level at about 7 m below the shoulder dimension. Important stabilisation works are therefore necessary.

The connection through line enters then an artificial bulkhead tunnel built with the "cut and cover"

*On 16 January 1991 the Major of Turin and the temporary Director of Italian Railways signed an agreement for the joint financing of Turin Railway Junction. This is meant to be the most important work to be carried out in the city during the last fifty years.*

*After decades of complete standstill from a town-planning point of view, when neither the underground network nor subterranean parking areas have ever been completed, and despite the particularly difficult time in the local and national economy, Turin now opens to a brighter future with the aid of this junction.*

*The building of the Junction assumes in fact an outstanding and decisive role in the future town planning of the Piedmontese city, by means of the so-called "Central Spine". This has been made possible by the running underground of the long town railway crossing, traditionally dividing Turin, in open cut and at level, longitudinally and side by side with the Po river.*



esclusivamente al traffico a media-lunga distanza consentirà l'instradamento in città della linea ad alta Velocità.

Il Nodo è costituito da un insieme di opere ferroviarie e di opere cittadine.

È importante sottolineare i concetti che sono alla base del progetto in esecuzione che, dall'originaria struttura degli anni '80, è stato completato con integrazione delle opere più propriamente ferroviarie con quelle urbane ricomprese nello schema preliminare del Nuovo PRG della città in corso di approvazione.

Infatti la costruzione delle nuove ferrovie è stata vista dal Comune di Torino come un'occasione per riqualificare l'ambiente urbano intorno alla ferrovia stessa. A tale scopo ha chiesto ed ottenuto dalle F.S. accollandosi tutte le maggiori spese, di costruire in sotterraneo le nuove linee e di coprire quelle esistenti e costruire sopra la ferrovia un grande viale urbano in maniera tale da utilizzare il sedime ferroviario per migliorare la viabilità, ricongiungere le parti della città precedentemente separate dalla ferrovia e valorizzare tutte quante le aree circostanti con un ridisegno della città.

Questo ha reso la costruzione dell'opera ferroviaria estremamente complicata ed ha posto una sfida piuttosto ardua ai progettisti che hanno dovuto conciliare le difficoltà costruttive con le limitazioni poste dall'esercizio ferroviario e dalla necessità di contenere al minimo i disagi per la città.

La realizzazione delle opere del Nodo si presenta a questo punto:

Il tratto Trofarello - Torino Lingotto è funzionante ormai da diversi anni. Il tratto Lingotto - To-Stura è stato iniziato nel 1986 con una prima parte a cavallo del fiume Stura di due chilometri e mezzo, tutto quanto allo scoperto, poi i lavori sono stati sospesi nel 1989 in conseguenza del riesame programmatico generale delle priorità degli interventi delle Ferrovie dello Stato e sono ripresi nel 1992; per la tratta funzionale Lingotto-To Porta Susa il cantiere ha incominciato ad essere operativo a giugno.

L'esecuzione è affidata alla Recchi S.p.A. come mandataria capogruppo che opera in associazione con le Imprese C.C.P.L., C.I.S. e FIAT Engineering.

## 2. Aspetti fondamentali delle strutture

Le strutture sono caratterizzate essenzialmente da due schemi principali:

— galleria naturale

— galleria artificiale sia per le nuove linee che per la ristrutturazione dell'esistente.

Va notato che tutte le opere sono fortemente influenzate dalla presenza dell'esercizio ferroviario e dagli esistenti manufatti sia per la ferrovia sia per le strade cittadine per cui esiste l'interferenza geometrica di vecchie e nuove strutture con necessità di interventi molto complessi che interessano di conseguenza anche nodi vitali di sopra e sotto servizi. Dal Lingotto, origine della tratta, per circa 1850 m il passante corre in trincea a lato dell'esistente linea Torino-Genova poi, approfondendosi, si inoltra in sotterraneo in galleria artificiale. In corrispondenza di Corso Bramante, che è una delle principali comunicazioni est-ovest della città, la linea si inserisce esattamente al di sotto della campata estrema del manufatto stradale sottopassandolo con il piano ferro a circa 7 m al di sotto della quota di imposta della spalla per cui occorrono importanti interventi di stabilizzazione.

Il passante prosegue in galleria artificiale a paratie eseguita con il sistema "cut and cover" da costruirsi in più fasi per mantenere l'esercizio della sovrastante linea Lingotto-Zappata e successive destinazioni. La quota del piano del ferro, continuando ad abbassarsi, incontra dapprima Corso Turati e quindi la zona di confluenza, verso il quadrivio Zappata, delle linee ferroviarie in esercizio. In questa situazione, così difficile nei confronti delle esistenze, il passante corre in galleria naturale per 352 m. A seconda delle situazioni geologiche-geotecniche e di sovraccarico dell'ammasso interessato, con particolare riguardo ai condizionamenti di superficie, sono state previste tre diverse metodologie di avanzamento.

Allo sbocco della nuova galleria il passante riceve al di sopra la linea veloce proveniente da Milano mentre a lato vi sono le opere esistenti del quadrivio Zappata. La struttura diventa un camerone con piedritti a paratie, complicato dalla presenza di una zona di servizio e di deposito di un gruppo scale di sicurezza e di un collettore fognario principale della città. Le paratie raggiungono la profondità dal piano campagna di circa 27 m.

In questa tratta e per una lunghezza di circa 250 m è inserita una fermata ferroviaria (Zappata) per cui la larghezza utile a piano ferro passa da quella standard di 11,40 m a 16; per le uscite di stazione sono inoltre presenti due zone con vani laterali per contenere i gruppi scale per cui la larghezza totale dell'opera in queste zone raggiunge i 24 m.

Dopo la fermata e fino in prossimità di Bivio Crocetta la struttura prosegue a due livelli ed è realiz-

zata lateralmente da setti di paratie mentre gli orizzontamenti sono costituiti da solette che chiudono la struttura a telaio.

Proseguendo la struttura interferisce con le trincee ferroviarie esistenti dell'attuale linea Porta Nuova - Porta Susa.

Le strutture sono coperte con travi prefabbricate in c.a.p.

All'altezza di Bivio Crocetta, si affianca al castello a 2 piani e alla linea lenta ristrutturata Porta Nuova - Porta Susa la nuova sede della linea per Modane. Dal Bivio Crocetta alle O.G.R. le strutture hanno la configurazione tipo illustrata nella sezione H-? In corrispondenza delle O.G.R., al fine di consentire al passante di instradarsi a Porta Susa dalla parte opposta a quella da cui proviene, inizia una complessa struttura tale da consentire al passante stesso di sottopassare con un flessio pianoaltimetrico la linea veloce Porta Nuova - Porta Susa e quella per Modane. La struttura è caratterizzata da un solettone intermedio a travi metalliche incorporate che sarà realizzato in due campi ai fini di mantenere, per fasi, l'esercizio: il primo campo verrà di seguito unito al secondo quando i treni verranno spostati. A questo punto del percorso i tre vani principali ferroviari raggiungono la stessa quota ed inizia la radice lato Lingotto della stazione di Porta Susa.

Nelle more dell'abbassamento della stazione è stato necessario studiare un tracciato pianoaltimetrico provvisorio delle linee per mantenere in esercizio la stazione di Porta Susa alla quota alta attuale, con alimentazione dalle linee già abbassate: il lavoro verrà svolto in più fasi.

La tratta termina 100 m circa oltre il C.so Vittorio Emanuele che viene sottopassato su una luce di 50 m circa con tre fornici di cui i due laterali da eseguire con scatolari a spinta di sezione di m 12 x 12 e lunghezza m 50 circa ciascuno.

## 3. Gli aspetti geologici e geotecnici del sottosuolo di Torino

L'infrastruttura ferroviaria da costruire si pone come un'opera che non ha precedenti per ampiezza nell'ambito della città, per cui non esisteva una quantità di dati sufficiente per una stima corretta dei parametri di base.

Il sottosuolo di Torino è costituito da un deposito sabbioso ghiaioso ben addensato con livelli cementati qualificabili come puddinghe per cui particolare importanza ha l'individuazione della esistenza, consistenza e permanenza di tali livelli sia per la gal-



stem. The tunnel will be erected in different stages maintain the over-passing Lingotto-Zappata line and subsequent destinations operational.

The rail level becomes lower and lower and meets at last with Corso Turati and the confluence area, towards the Zappata cross-roads of operating railway lines. In this situation, so difficult for existing lines, the connection through line runs through a natural tunnel for 352 meters. Following the geological geo-technical features and the affected shoot overload, with special consideration for surface constraints, the following three advancement methodologies have been envisaged:

At the exit of the new tunnel, the connection through line is linked above with the fast line from Milan and runs parallel to the Zappata cross-roads. The structure becomes a big chamber with bulkhead piers, complicated by the presence of a service and deposit area, by a set of safety stairs and by the city sewage system main manifold. Bulkheads reach a depth of about 27 meters from the plane of site. Along this route section and for about 250 meters, a railway stop (Zappata) has been introduced; the useful length at rail level moves therefore from the standard 11,40 to 16 m; for station exits two supplementary areas have been envisaged sideways to host the stairs. Total work length in this area reaches therefore 24 meters.

After the stop and up to close to Bivio Crocetta the structure proceeds at two levels and consists of side bulkheads and horizontal slabs closing the frame structure.

Then the structure interferes with the existing cuttings of the current Porta Nuova - Porta Susa line.

Structures are covered by pre-fabricated beams in precompressed reinforced concrete.

To Bivio Crocetta the two-floor castle and the restructured slow Porta Nuova - Porta Susa line are joined by the new Modane railway line.

From Bivio Crocetta to O.G.R. (Major Repair Workshop), structures have the typical configuration illustrated in section H.

At the O.G.R., for the connection through line to be routed to Porta Susa from the opposite direction from where it comes, a complex structure starts that enables the connection through line to underpass the fast Porta Nuova - Porta Susa line and the Modane line with a planimetric-altimetric inflection. The structure is composed of an intermediate slab with incorporated metal beams. It will be built in two fields to maintain railway operation: the first field will be then added to the second one only once trains are transferred.

At this location of the path the three main railway openings reach the same elevation and the Porta Susa station roof by the Lingotto side starts.

Awaiting the forthcoming station lowering, a provisional planimetric-altimetric train path had to be worked out to maintain the Porta Susa station operational at the current high elevation, with feeding from already lowered lines: this work will be carried out in several stages.

The route section stops at 100 meters beyond Corso Vittorio Emanuele that is underpassed with a 50 m span approximately with three frames; the two side structures are to be carried out with box culverts, each with a 12 x 12 section and 50 m length approximately.

### 3. Geological and geo-technical features of the Turin underground

The railway infrastructure to be built is unprecedented in terms of size and extension in the city, therefore existing data available were insufficient for a correct evaluation of the main parameters.

The Turin underground consists of a well densified sandy gravelly deposit with cemented seams that can be qualified as puddingstone. Therefore it is particularly important to identify the existence, consistence and permanence of these seams both for digging and consolidation methods of the blind tunnel and for bulkheads digging.

Physical and mechanical features of ancient floods relevant to soil formation (Fig. 4) are not just linked to the periglacial or fluvial deposits they stemmed from, but also to the action of underground water of a different origin and different ionic content flowing in the subsoil. The confluence and mixing of hard and cold water flows coming from the Alps to the West, with milder waters with a different pH led to the precipitation of ballstone, gradually less massive and continuous, proceeding in the city area from North-West towards South-East. Hence random ballstone distribution, frequency and extension that made a study essential to carry out the excavations and define construction techniques.

After a first general geological study of the whole train path, it was therefore necessary to hold an in-depth survey relevant to the route sections subject to construction.

These surveys encompassed all classic techniques, then supplemented by the test building of over 10.000 square meters of bulkheads with different

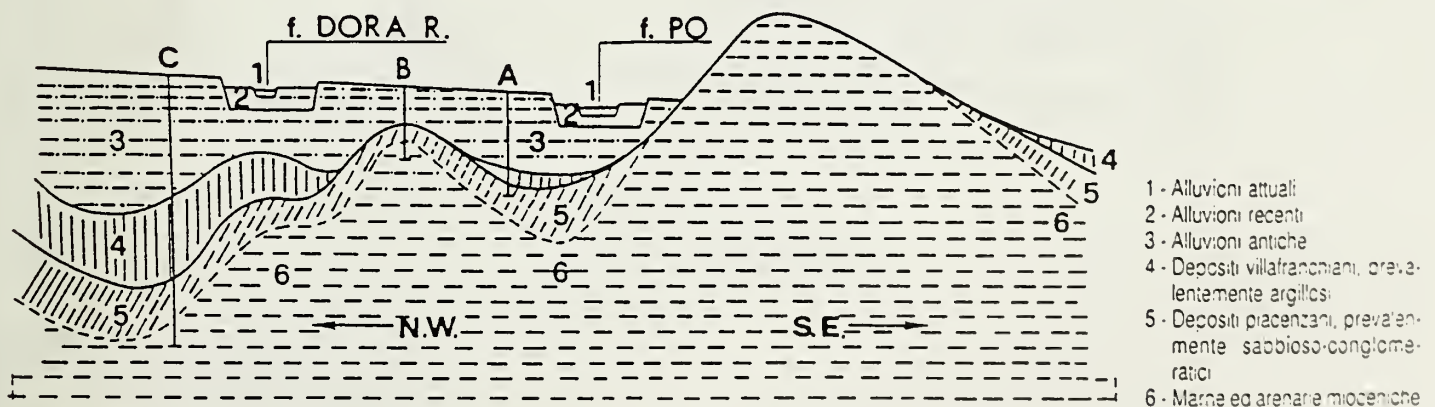
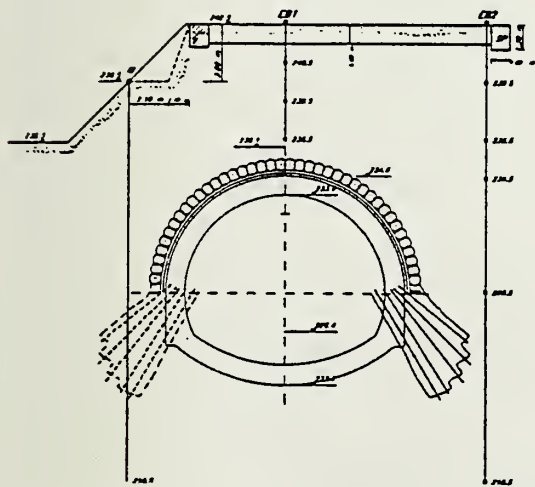
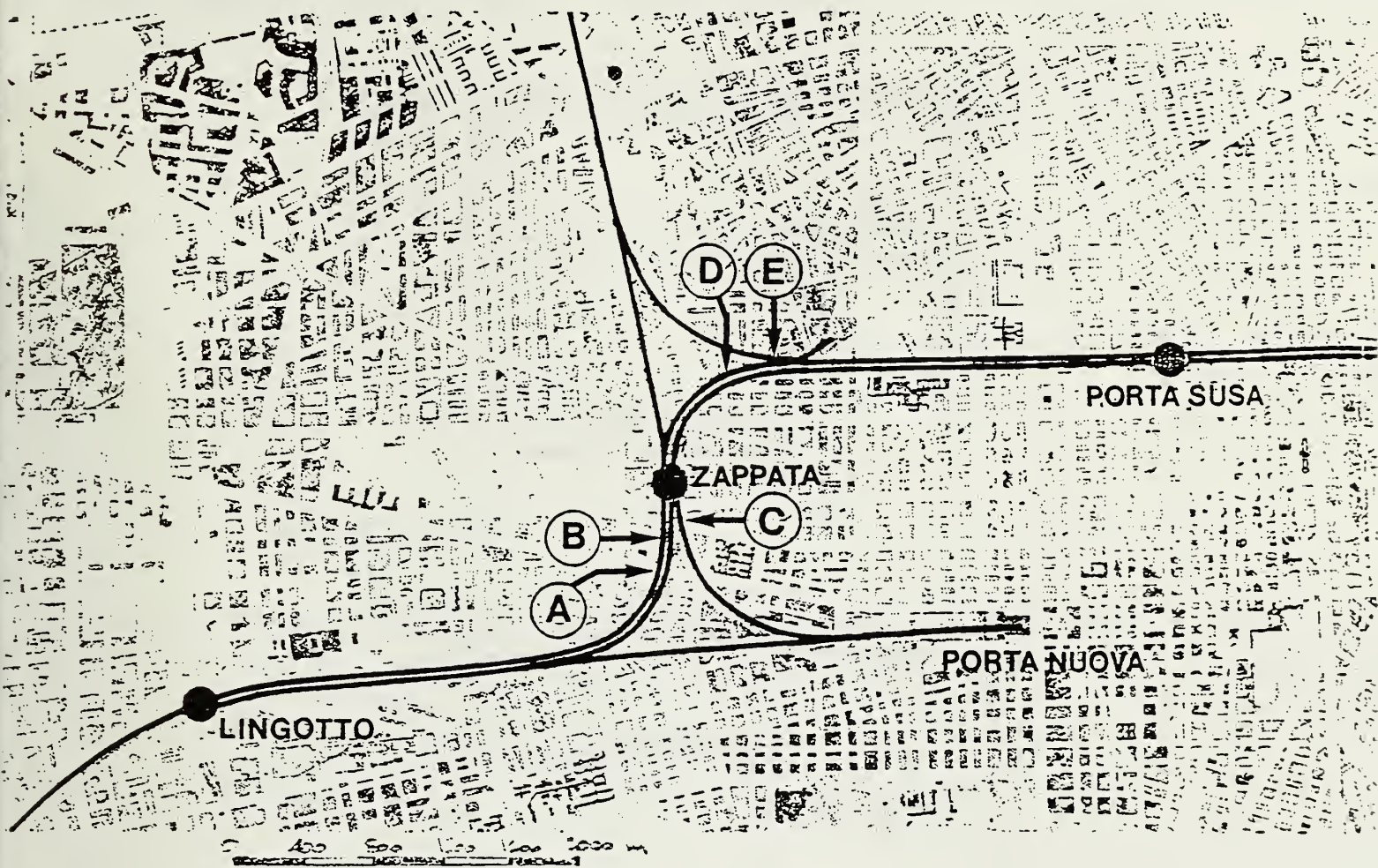
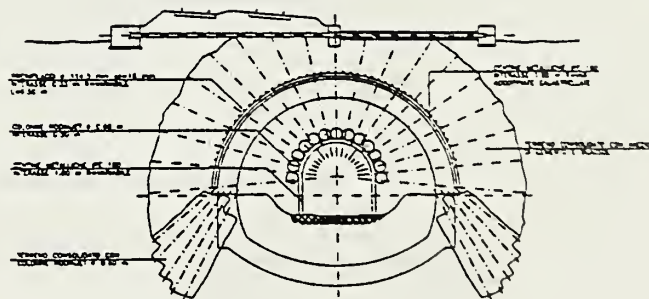


Fig. 2 - Sezione geologico-tecnica schematica della zona tra i fiumi Dora Riparia e Po nell'area urbana torinese  
1-2 - Two Recent Alluvium Deposits; 3 - Ancient Alluvium; 4 - Villafranchian Deposits, mainly clay; 5 - Piacenzian Deposits, mainly sandy conglomerate; 6 - Marl and Sandstone (Miocene)  
Geological Cross Section of the area between Dora Riparia and Po rivers in the city of Turin





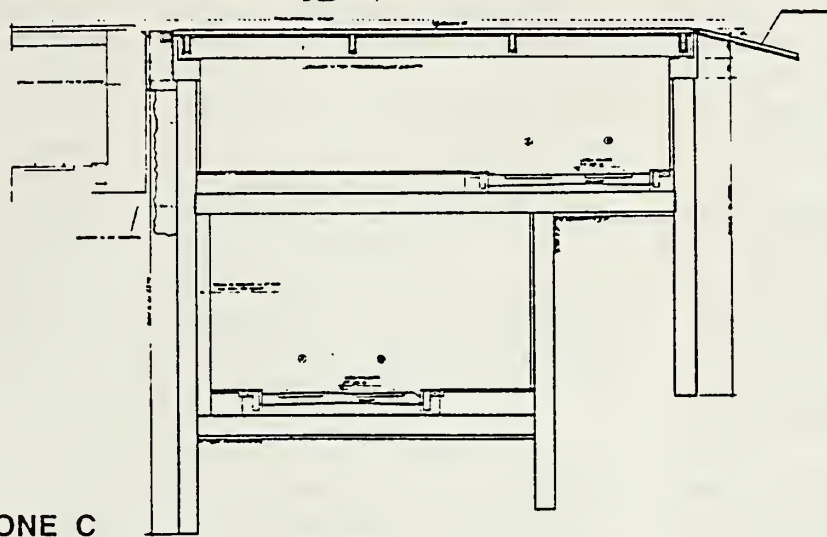
SEZIONE A



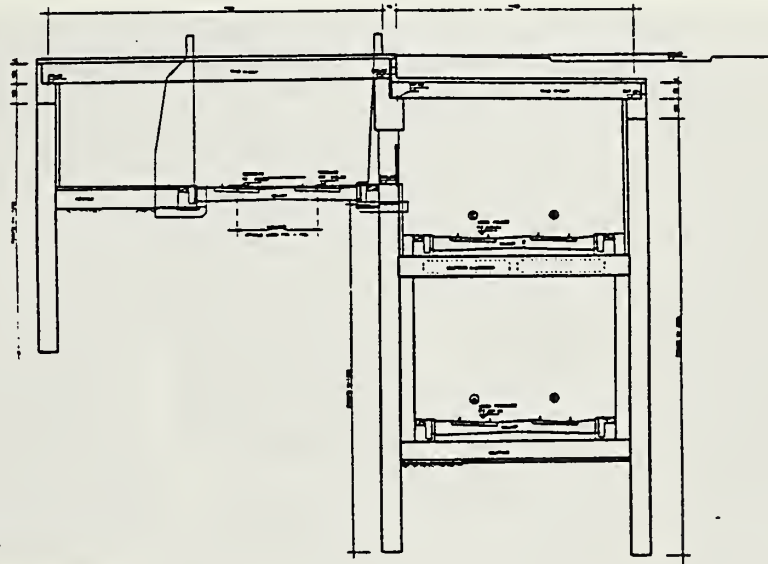
SEZIONE B

Pianimetria della prima tratta funzionale del nodo di Torino  
in costruzione e principali sezioni tipologiche  
Site plan of the 1<sup>st</sup> functional route of the Turin railway junction  
currently under construction and main section types

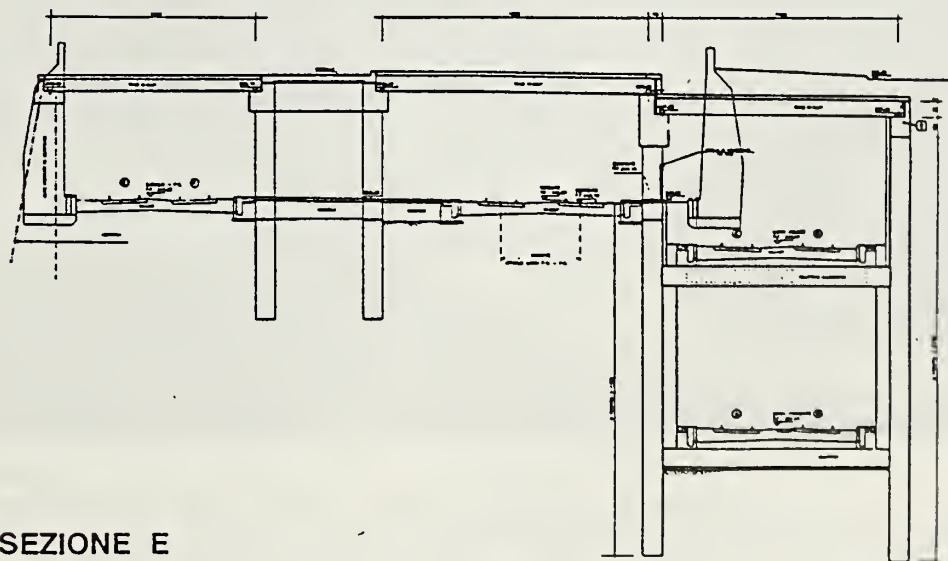




SEZIONE C



SEZIONE D



SEZIONE E



era a foro cieco, metodi di scavo e consolidamenti, sia per le tecniche di scavo delle paratie. Le caratteristiche fisico-meccaniche delle alluvioni antiche costituenti la formazione (fig. 4) dipendono non solo dall'ambiente deposizionale-periglaciale e fluviale in cui si sono originate, ma anche dall'azione delle acque sotterranee, di diversa provenienza e contenuto ionico, che le hanno percorse. In particolare, l'incontro e la miscelazione nel sottosuolo di acque dure e fredde provenienti dalle Alpi, a Ovest, con altre più temperate e di diverso pH, ha dato luogo alla precipitazione di concrezioni calcaree, in forma via via meno massiva e continua, procedendo nel territorio della città da Nord-Ovest verso Sud-est. Ne risultano distribuzione, frequenza ed estensione delle concrezioni del tutto casuali e l'indagine ad esse relativa si poneva nei confronti degli scavi come fattore fondamentale per l'impostazione generale delle tecniche costruttive.

È stato così necessario eseguire, dopo un primo inquadramento geognostico generale dell'intero tracciato, una campagna molto approfondita di indagini per le tratte oggetto della fase funzionale costruttiva.

Tali indagini hanno ricompreso tutte le tecniche classiche che sono state completate con le costruzioni in prova di oltre 10.000 mq di paratie con diversi utensili di scavo nonché con l'esame di cunicoli anch'essi in fase di avanzamento.

Questi dati sono stati paragonati con il metodo di sondaggi PA.PE.RO. che consiste nella determinazione del valore dell'energia specifica spesa da un utensile nel corso della perforazione del terreno naturale.

Il parametro energetico si dimostrava infatti determinante per evidenziare le variazioni, lungo la profondità, delle caratteristiche geotecniche e di scavabilità del terreno, e per tarare tutto il complesso

dei dati.

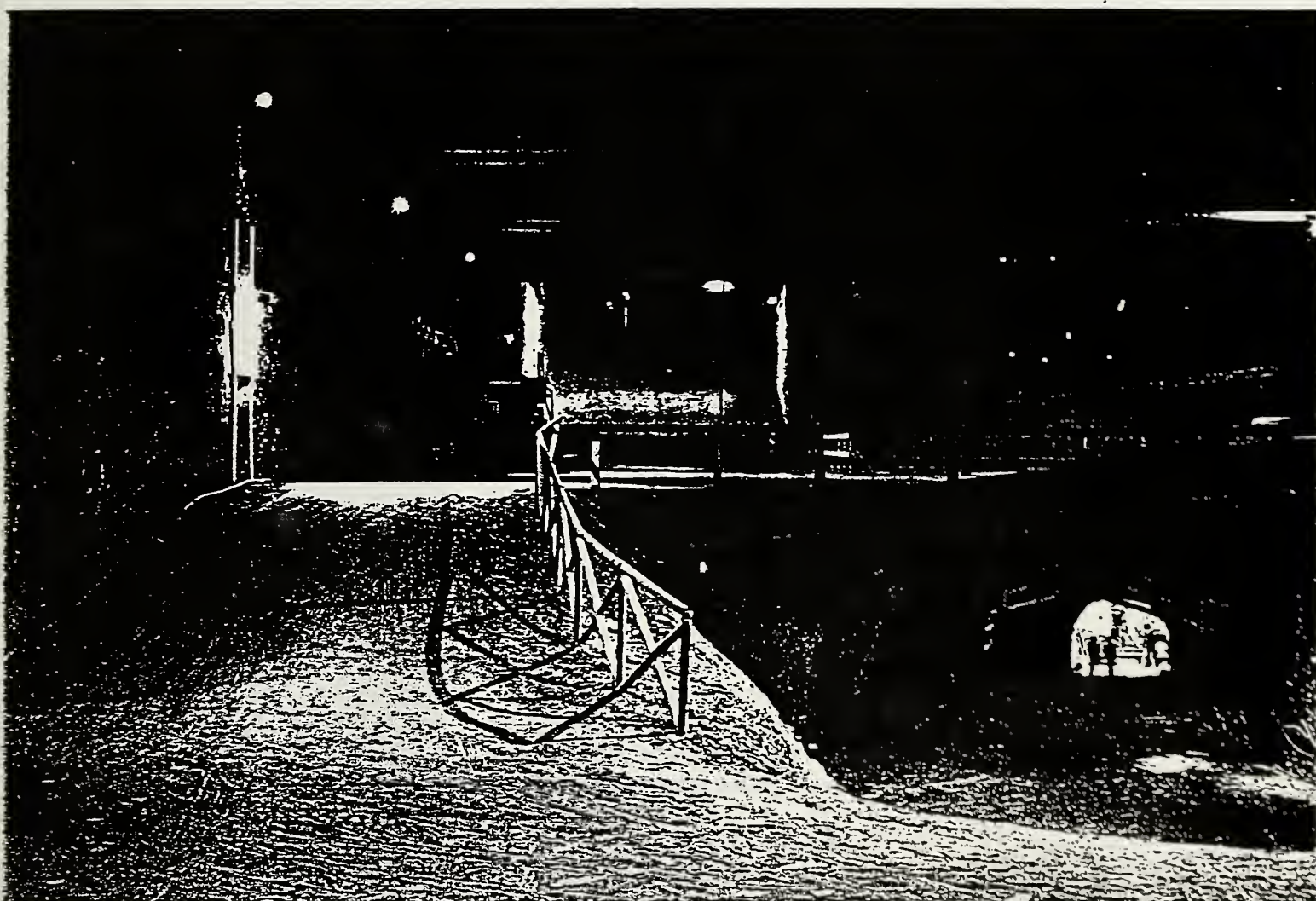
L'energia specifica (E) è calcolata in funzione della spinta (F) sull'utensile inserito nel foro di sezione (A) ed avanzante con velocità di rotazione (N) e velocità di avanzamento (u) sotto una coppia applicata (T) ed è espressa da:

$$E = F/A + 2 \pi \cdot (N \cdot T/u)/A \quad (\text{kJ/m}^3)$$

I parametri valutati e correlati con tutto il complesso delle altre indagini hanno indotto a definire due differenti campi:

- il campo del terreno "scavabile" con benna, dove E è inferiore a 1.000.000 kJ/m<sup>3</sup>;
- il campo del terreno "non scavabile" con benna, dove i valori di E sono superiori a 1.000.000 kJ/m<sup>3</sup>;

Il criterio di scavabilità fa riferimento inoltre alla effettiva continuità di presenza (persistenza), dei valori elevati; persistenza significativa è stata consi-



Nodo di Torino - Camerone di confluenza

Sotto: La linea passante asse principale del progetto - Sopra: La nuova linea veloce di quadruplicamento dell'esistente linea

Below: The new railway "connection through line" forming the main axis of the project - Above: The track doubling of the present line forming a new "high-speed" route partially overlapping



digging tools, and by the analysis of underground passages under construction.

These data were compared with the PA.PE.RO. boring method that envisages the calculation of specific energy used by a tool during digging in natural soil.

The energy parameter was indeed essential to highlight variations of the soil geo-technical and digging features at different depths and to calibrate the whole set of data.

Specific energy (E) is calculated as a function of thrust (F) on the tool inserted in the section bore (A), moving forward with rotation speed (N) and feed speed (u) under an applied torque (T) and expressed by:

$$E = F/A + 2 \pi \cdot (N \cdot T/u)/A \quad (\text{kJ/m}^3)$$

These parameters evaluated and correlated with the results of the other surveys led to the definition of

two different fields:

- the field of soil that could be dug with a bucket, where E is lower than 1.000.000 kJ/m<sup>3</sup>;
- the field of soil that could not be dug with a bucket, where E values are higher than 1.000.000 kJ/m<sup>3</sup>;

the «digging» criterion makes reference to the actual persistence of high values; significant persistence was considered for thickness values exceeding 10 cm.

The pair of threshold data relevant to energy and persistence exceeded 40% of data thereby allowing for the definition of a digging profile that became the basis for subsequent technological decisions relevant to the project.

#### 4. Industrial approach to the construction problem state-of-the-art

##### 4.1. The natural tunnel

— Technological approach to the construction problem

With reference to geometrical constraints and in compliance with the nature of the Turin soil, specific consolidation techniques have been envisaged to carry out a highly resistant treatment in a limited arch around the digging profile. Where possible, priority was therefore attached to jet-grouting, to confer high resistance in small soil volumes, versus injection that produces modest resistance with high volumes. Measures were taken to avert the danger of lifting by claquage (especially considering the presence of operating railway lines in the vicinity), and of syneresis. The injection of non-polluting material was unavoidable in cases of very limited coverage, where reinforcement with metal pipes in special arrangements was necessary. This was intended to confer homogeneity to the shoot and compensate for its heterogeneity.

##### 4.2. Artificial tunnels with brattices

— Digging with the «hydro-mill» unit

For the first time in Italy a special technique was successfully adopted; it envisages the joint use on each vertical plate element of two coupled tools, one with milling heads and inverted circulation of bentonitic mud.

The milling equipment called «hydro-mill unit» was only recently developed and applied abroad where it became productive on an industrial scale.

The hydro-mill is made of three main parts: the supporting excavator, the milling unit and the vibratory screen.

For the Turin Railway Junction, given the above-mentioned soil constraints, an international survey was carried out to identify the 5 best-suited types of machines that will be used in the Junction. The presence of cemented layers alternated with dissolved flowages restricted the choice of the digging methodology.

Puddingstone has the appearance and consistence of concrete with inert made of sand and gravel, with round pebbles and calcite as binder.

Cementation strongly reduces the performance of clamshell buckets normally used to build brattices, because it prevents their teeth from penetrating between gravel and pebbles and considerably limits their mobility when the bucket clamshells manage to grasp them.

Thanks to rotating tools equipped with teeth and special bits, calcite is ground under rotation and this process induces the rolling of gravel and pebbles torn off the gangue.

In case of large pebbles, to avoid waste of energy for shredding and allow for subsequent mucking intake, normal clamshell buckets can be utilised. In a subsequent note the detailed work stages and the machines chosen will be described and the relation between plan and operational results will be stressed with special reference to the nature of material and to the energy problems cropped up during excavations.

The main operating sites were opened starting from June 1992 and are now twelve:

- 2 digging sites for the natural tunnel
- 5 for bulkheads
- 2 for special consolidations
- 3 for moving sewage manifolds and underground utilities of primary importance.

Others will be opened soon to carry out the remaining works and attain total plan implementation by 1996.



the "connection through line"



derata per spessori superiori a 10 cm.

La coppia dei dati limite relativi alla energia e alla persistenza ha superato il 40% dei dati consentendo di costruire un profilo di scavabilità che è stato la base delle scelte progettuali tecnologiche.

## 4. L'approccio industriale del problema costruttivo e stato dell'arte

### 4.1 La galleria naturale

L'impostazione tecnologica del problema realizzativo tenuto conto dei vincoli geometrici e in coerenza con la natura del terreno di Torino sono state prese tipologie di consolidamento particolari tali da realizzare un trattamento molto resistente entro un arco molto raccolto intorno al profilo di scavo. Si sono privilegiati quindi, ove possibile, interventi con grouting, capaci di dare resistenze elevate in piccoli volumi di terreno, rispetto a interventi con iniezioni, che producono invece resistenze al confronto mediocri, su volumi estesi; si è cercato di evitare in tal modo il pericolo di sollevamento per claquage (e ciò in particolare per la presenza di linee ferroviarie limitrofe in esercizio) nonché i pericoli di sineresi. Le iniezioni di materiali non inquinanti sono state indispensabili in situazioni di ridottissima copertura dove è stato necessario integrarle con infilaggi di tubi metallici variamente disposti. Questi ultimi interventi sono intesi a dare omogeneità all'ammasso per sopperire alle sue caratteristiche di eterogeneità.

### 4.2 Le gallerie artificiali con diaframmi

Lo scavo con il gruppo idrofresa

È stata positivamente adottata per la prima volta in Italia una tecnica che prevede l'impiego congiunto, su ciascun elemento di setto verticale, di due attrezzature accoppiate, una a teste fresanti con circolazione rovescia di fango bentonitico.

Le attrezzature con fresa, denominate "gruppo idrofresa" sono state messe a punto di recente e sono operanti all'estero dove hanno raggiunto produttività su scala industriale.

L'idrofresa è costituita da tre parti sostanziali: l'esca-

vatore di sostegno, il gruppo fresante e il vibrovaglio. Per Torino, stante le condizioni di vincolo al contorno, sono state indagate in campo internazionale le caratteristiche delle macchine esistenti che sono state individuate in 5 tipi tutte quante scelte per operare al Nodo.

La presenza di alternanze di strati cementati e di alluvioni sciolte, ha condizionato la scelta della metodologia di scavo.

La puddunga si presenta con l'aspetto e la consistenza di un calcestruzzo con inerte costituito da sabbia e ghiaia con ciottoli rotondi e calcite per legante.

La cementazione influenza pesantemente il rendimento delle benne mordenti normalmente usate per la costruzione dei diaframmi, perché impedisce la penetrazione dei denti fra gli elementi ghiaiosi ed i ciottoli e ne limita considerevolmente la mobilità quando anche le valve della benna riescono in qualche modo ad afferrarli.

Con l'adozione di utensili rotanti muniti di denti e scalpelli opportuni la calotte viene macinata sotto l'azione di rotazione che trascina in un moto di rotolamento ghiaia e ciottoli divelli dalla matrice.

In caso di ciottoli di grosse dimensioni, per evitare perdite di energia per la loro frantumazione e rendere così possibile la successiva aspirazione per lo smarino, si può convenientemente usare in ausilio la normale benna mordente.

In una successiva nota si descriveranno in dettaglio le sezioni delle opere e le macchine scelte e si porrà in rilievo il rapporto tra progetto e i risultati operativi con particolare riguardo alla natura dei materiali e ai problemi energetici emersi durante gli scavi. I cantieri operativi principali aperti dal giugno '92 ad oggi sono in numero di dodici:

— 2 di scavo per la galleria naturale

— 5 per le paratie

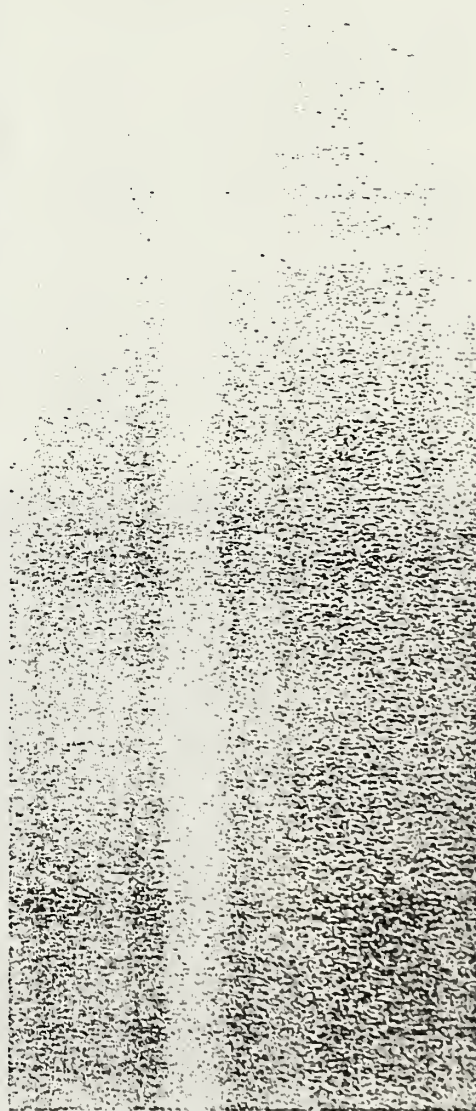
— 2 per consolidamenti speciali

— 3 per gli spostamenti di collettori fognari e sottoservizi di primaria importanza;

altri saranno aperti a breve termine per gli altri interventi in sincronia con il programma che prevede l'ultimazione delle opere nel 1996.

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# Jet setting under Bonn

*Fiona McWilliam, Assistant Editor*

Numerous restrictions hamper the excavation of tunnels under urban areas. In Bonn, a contractor driving a 497.5m-long tunnel using the NATM, in ground pre-treated with jet grouted piles, faces many constraints. The problems are compounded by an extremely shallow depth of cover. Nevertheless, the 7.8m-high D-shaped tunnel is progressing well, despite stretching Austro-German contractor Beton- und Monierbau's tunnelling capabilities to the limits.

**T**his is the second part of Beton- und Monierbau's contract at Bonn. It will eventually form part of a 2.4km-long, predominantly cut-and-cover tunnel to carry the Bonn-Bad Godesberg two-track metropolitan tramline beneath its original surface route. The contractor for the entire project, a seven-company JV led by Hochtief, awarded a contract to Beton- und Monierbau in 1990 to carry out the mining part of the scheme. According to site manager Johann Herdina, this was at the insistence of client Bonn Metro, in acknowledgement of Beton- und Monierbau's experience with the NATM.

The company, a subsidiary of Deilmann-Haniel, has already driven a 150m-long tunnel as the first phase of the contract (*T&T*, Oct '90, p19). This was excavated by side wall drift, a method specified by Deutsche Bundesbahn, under whose main Cologne-to-Frankfurt line the tunnel passes.

A very different approach to excavation exists at this tunnel. It involves the pre-treating of the ground with a canopy of jet grouted piles, occasioned by several factors, not least poor geology and the necessity of avoiding disruption to overhead tram and road traffic.

## Poor tunnelling medium

The tunnel is being driven through irregular layers of loose, unconsolidated, 'rolling' Rhine gravels with a varying permeability averaging 0.008m/s and containing irregular sand layers. From the southern portal, layers of silt are encountered in the crown for approximately 190m.

"It is a very poor tunnelling medium and tricky to handle", explained Herdina. "The groundwater level is high and there is only a thin layer, approximately 14-15m, of workable ground." For 400m, including 250m of the tunnel, the water table is lowered by 1 to 1.5m.

The depth of cover for the 7.8m tun-

nel averages 3.5m, and several metres in from the north portal, where cover is at its minimum (3.3m), this overburden is intersected by a mains sewer (Fig 2).

But these are not the only factors to complicate the project. The location of the tunnel between the federal capital and the main residential town of Bad Godesberg severely limits the site area and storage facilities are minimal. Each portal is located within an unfinished cut-and-cover station section. Adjacent to the north portal, overhead tram lines are supported on large steel columns. If any equipment were to crash into one of these columns, the results could be fatal. There is, however, enough room outside each portal for a compressor and pressure pumping equipment for shotcrete. Prepared with water from the dewatering system, it is applied at a pressure of 7 bar. These facilities are crucial, since work with the NATM has to be continuous to minimise settlements, and no nearby mixing station is permitted to

operate 24h. day.

In addition to passing almost directly beneath the existing tram lines, the tunnel is crossed close to the north portal by the northbound two-lane carriageway of the B9 (Fig 1). The B9 is the only main road between Bad Godesberg and Bonn, and, like the tram, is a major commuter link.

"The B9 carries approximately 30 000 cars/day, but I'm sure that if people realised the size and proximity of the tunnel, there wouldn't be any traffic at all", commented Herdina.

The City of Bonn's choice of jet grouting in conjunction with the NATM was prompted, said Herdina, by more than the ground conditions. Bonn hopes to gain experience for a future project to lay the B9 underground beneath the current works. Beton- und Monierbau's experience in ground preparation by jet grouting includes a similar project carried out recently in Austria.

Excavation started from the south



*The SR510 Rodinjet machine working at the north face of the tunnel.*



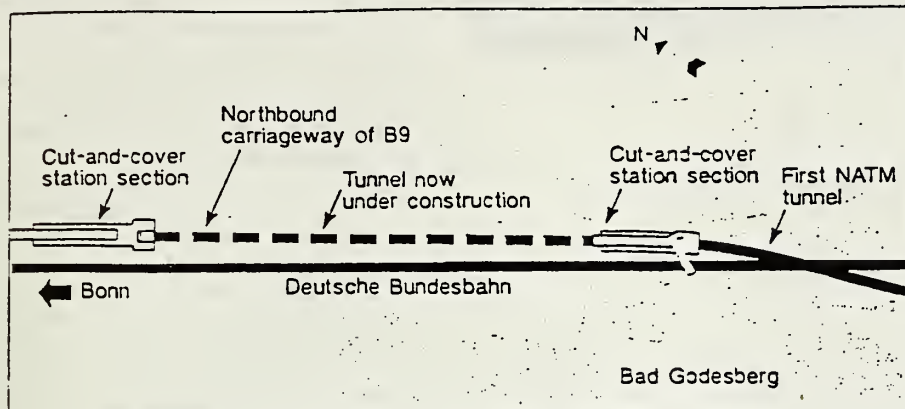


Fig 1. Alignment of the new tunnels, beneath the existing metro line.

portal on Nov 26, 1990, and from the north portal on Dec 5. Beton- und Monierbau expects to have completed its part of the contract by July 1991, after which the main contractor will carry out internal lining and track laying.

## Two faces

The tunnel is being driven from both sides by a piling crew of five men subcontracted to Beton- und Monierbau from a JV comprising Keller and Rodio group company Eurosond, and a five-strong Beton- und Monierbau mining crew. As the piles are too soft to be excavated under immediately after they are placed, the crews alternate between the two faces, using similar sets of jet grouting and excavation equipment.

Two 19.6m-long Rodio SR510 Rodinjet

machines are used to place the 12m-long piles around the perimeter of the shotcreted faces. Each of these drills a 114mm-diameter steel lance into the face. As this is withdrawn, a 1:1 ratio cement water mix is jetted into the drilled cavity which forms the 600mm diameter piles. These secant piles, at 470mm centres, are jetted at an angle of approximately 10°, creating a 3m overlap with the preceding set of piles.

In the north portal, the general procedure is that 37 piles are driven at a 4.6m radius around the top and sides of the face. It takes three days for all these to be driven, after which the piling crew moves to the opposite portal, enabling the mining crew to start work on the face.

Three days of excavation advances the

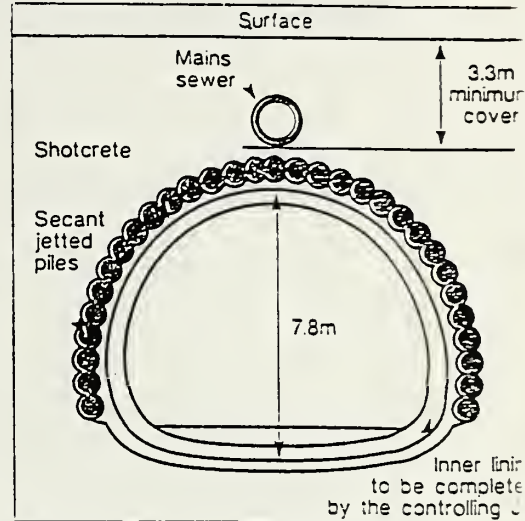
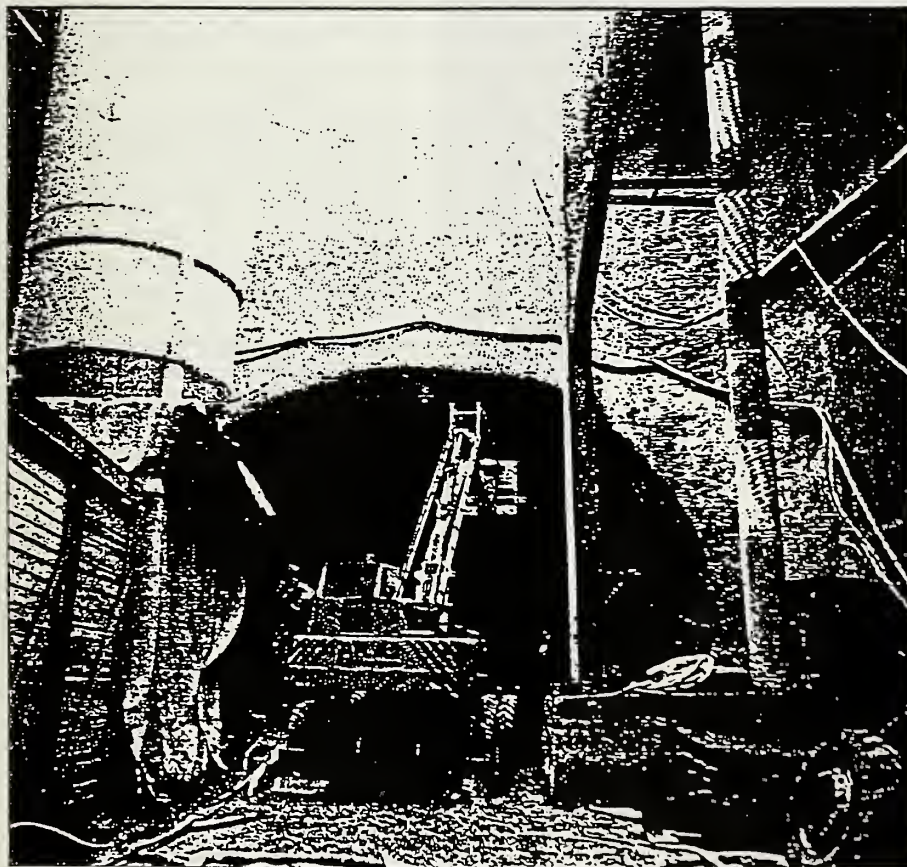


Fig 2. Sectional view of tunnel.

face by 9m. The procedure involves the excavation of a 4m bench and the removal of the invert 3m behind, in 3m sections. A 'nose' of material is left in front of the face in order to prevent slippage from the top (Fig 3).

After every 1m advance in the crown, a 30 to 50mm layer of shotcrete is applied to all open areas. Two sections of steel arch are then erected and fitted into 1m-long horizontal steel footplate sections. After this, a steel wire mesh is attached to the back of the arch and a 150mm-thick layer of shotcrete is applied to it, providing a surface flush with the arch. A second wire mesh is then applied to this surface, followed by a final layer of about 50mm of shotcrete. Each completed ring comprises five arch sections: the upper four follow a radius of 4.6m and the final one in the invert has a 15.7m radius.

A Demag 41 track-mounted roadheader is used initially to smooth the inner surface of the piles forming the crown, while the actual excavation is carried out by an Atlas 1602EK swivel-head backhoe excavator. Liebherr 531 loaders are used to remove muck, some of which is shipped down the Rhine by barge to the Netherlands, where it is used as an aggregate. Much is retained



View of the north portal with the existing metro line on steel columns.

**Name of project:** Bonn Metro Phase II (second NATM tunnel)  
**Client:** City of Bonn  
**Consultant:** Obermeyer and Beton- und Monierbau  
**Contractor:** Beton- und Monierbau subcontracted to a JV of seven companies led by Hochtief  
**Excavation method:** Atlas 1602EK backhoe excavator  
**Length of tunnel:** 497.5m  
**Average tunnel dimensions:** height (to steel arches): 7.8m  
**Excavated width:** 9m  
**Cost:** Entire project \$63.9m  
 NATM tunnels \$13.4m  
**Time frame:** 192 days



locally, however, for infilling the cut-and-cover sections.

At the time of *Tunnels & Tunnelling's* visit in late February 1991, only 20 piles were being used to advance the south face, ten either side of the crown section supported by rebars. The crown was at this time located within self-supporting silts, and was expected to encounter these for another 49m.

The 4m-long steel bars supporting it are spaced at 350mm intervals, and inserted by a narrow forepoling rig custom-built by Beton- und Monierbau for side wall drift working in the earlier tunnel. Irregularities on the inner surface of the piles at the south face are removed by hydraulic hammer, but excavation and mucking involves the reuse of equipment employed at the opposite side.

The presence of a second drain, approximately 13.5m in from the north portal, dictated a third method of crown support - steel plates. These were used for a 12m length.

Three fixed laser beams are used to guide the progress of the tunnel, which is virtually straight and horizontal over its entire length. Due to the positioning of the piles, which radiate outwards to prevent the tunnel becoming smaller in section with each jet grouting stage, the sides of the tunnel expand in 9m-long steps (Fig 3). Beton- und Monierbau is currently constructing shuttering at each portal which will be used to apply concrete to the tunnel profile.

## Monitoring movement

As with the earlier tunnel, minimising settlement above is crucial and this in turn depends on detailed and accurate ground monitoring. Beton- und Monierbau employs a resident surveyor on site, who currently monitors 70 out of a total of 274 surface points daily. The rest are checked weekly or monthly, although this frequency will be increased should difficulties arise. The surveyor has on-site access to the company's computerised surveying system, which provides print-outs of point movement to an accuracy of  $\pm 1\text{mm}$ .

Within the tunnel, a set of five circumferential diodes at 18m centres are monitored daily by theodolites placed at fixed points along the walls. This monitoring is vital as it dictates rapid response to ground movement.

According to Herdina, jet grouting can minimise surface settlement to between 50 and 60% of that experienced with the side wall drift method: "We were originally worried about the possibility of settlement due to material being washed out by drilling, but this caused no problems, probably because of the minimal time lapse between drilling the holes and jetting." Although some settlement is inevitable, the average recorded is just 10mm.

The initial jetting of the pile at a

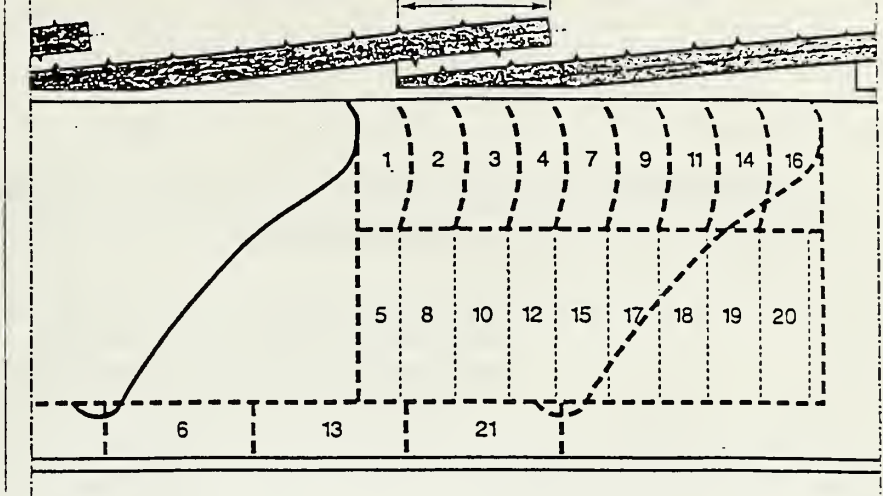


Fig 3. Excavation procedure showing how invert is removed in 3m-long sections.

pressure of 400 bar, using 250 litre/min of the cement/water mixture, resulted in a very different problem, that of uplift by as much as 91mm.

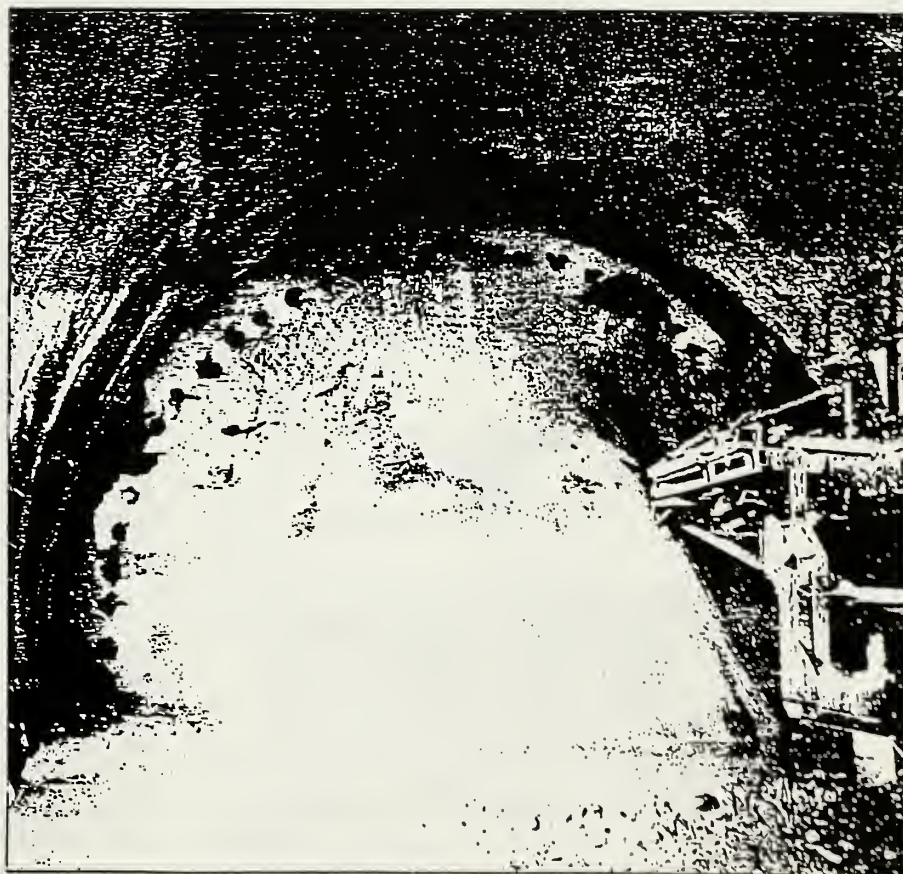
## Counteracting uplift

This is believed to have been caused by the 10% angle of the jet grouting into virtually horizontally bedded material. The grout, unable to flow into the sand layers, 'cushions' outwards into the loose gravel, forcing the sand layers apart.

"The logical reason for this was that we were blasting too much material at too great a pressure, so we went down to 300 bar," explained Herdina. "However,

this was not sufficient and 30mm uplift still occurred." The problem was eventually overcome by drilling a series of pressure-release holes into the crown above every other jetted pile, which reduced uplift to approximately 8mm. Combined with the minimal but inevitable settlement, the overall movement caused by the excavation of the tunnel balances out to an average of approximately 2mm of settlement.

The completion of this tunnel is set for the end of July 1991 and, with problems successfully overcome, the project is on schedule and within budget.



Shotcreted face with 'nose', awaiting excavation by the mining crew.



# Sub-horizontal jet grouting applied to a large urban twin tunnel in Campinas, Brazil

## 'Jet-grouting' subhorizontal appliqué à un double tunnel de grandes dimensions à Campinas, Brésil

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**SYNOPSIS:** Jet-grouting represents one of the most recent developments in injection techniques. The outstanding feature of this method is the ability to treat a wide range of soils regardless of their permeability or structure, by use of a simple cement grout mixed in place with soil particles under a very high injection pressure. Tunnelling is, at present, the main field of application of this technique.

RODIO in 1983, first in the world, applied jet-grouting in tunnels in a sub-horizontal way for soil consolidation ahead of the excavation face. This new technique was first introduced in South America in 1987 by RODIO-NOVATECNA Joint Venture for the excavation of a twin shallow large-size tunnel in CAMPINAS (Brasil).

This paper, following an introduction to the general features of the technique, outlines this specific case history.

### INTRODUCTION

The construction of underground openings may require preliminary ground treatments, which can be particularly useful in cohesionless or weak cohesive soils, especially in urban areas to protect the integrity of buildings and services.

The most known techniques of soil improvement for tunnelling are connected to the injection of grouts (for permeation at controlled flow-rate and pressure or by hydrofracturing in case of low-permeability soils) and to the temporary soil freezing, of which its most interesting and competitive application is the consolidation of fine-grained soils at lower limit of injectability. These two techniques are limited by the soil permeability to grouts, in the case of injections, and by the high cost in the case of soil freezing (the application of the latter is generally restricted to the most complex and critical cases). The great interest for the new ground improvement techniques based on jet-grouting is due to its capability of cutting soil using a high pressure jet, optimizing the injection energy and the uniformity of the treated soil independently from the soil permeability to grouts. A high and rather uniform improvement in shear strength can thus be reached inside the treated bodies.

In 1983 RODIO first applied the jet-grouting technology for sub-horizontal soil consolidation ahead of the excavation face in tunnelling. This technique is presently applied together with the New Austrian Tunnelling Method, to half section excavation, in shallow tunnels in soil.

Horizontal jet-grouting can be applied advantageously when the surface is not accessible, when existing roads or railways have to be crossed, in urban inhabited areas, and in detrital soils generated by landslides.

### TECHNOLOGY AND EXECUTION METHOD

The jet-grouting is an injection of cement grout in the soil at extremely high pressures (30-60 MPa) through small diameter nozzles (2 to 4 mm) located normally to the drilling axis. The fluid jet coming out of the nozzles at high velocity, cuts the surrounding soil and simultaneously mixes it in place with the stabilizing grout. Columns of cemented soil are then formed, the diameter of the treated soil generally varies between 0.4 and 0.7 m, depending essentially on the injection para-

eters and natural soil strength (mainly cohesion).

Column diameter can be incremented up to over 2 m, using systems with two or three fluids (grout jet - water jet + compressed air), anyway the higher flexibility of the single fluid method "CCP-RODINJET" facilitates its application to the sub-horizontal treatments.

In fig. 1 the execution sequence is shown together with a general layout of the geometrical scheme. Columns of jet-grouting are realized ahead of tunnel face, in a conical geometry layout. The group of overlapping sub-ho-

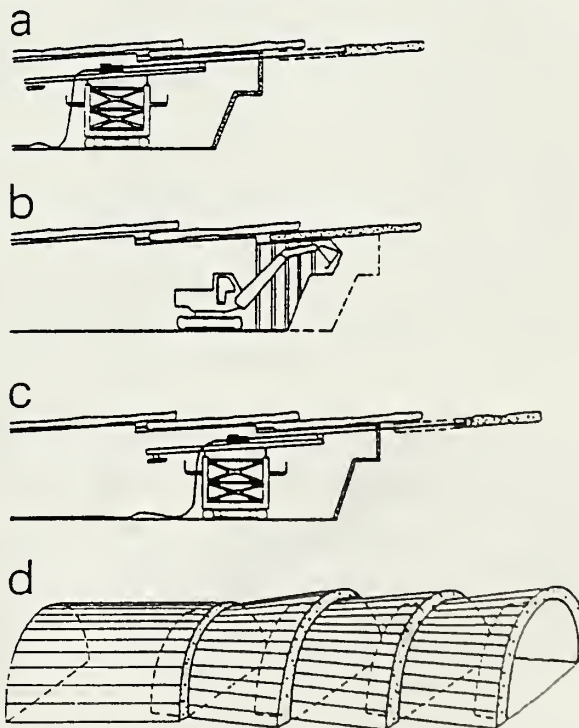


Figure 1. Sub-horizontal jet-grouting execution sequence  
 a) jet grouting ahead of the face - b) excavation  
 c) jet grouting at next stage - d) geometrical scheme of treatment



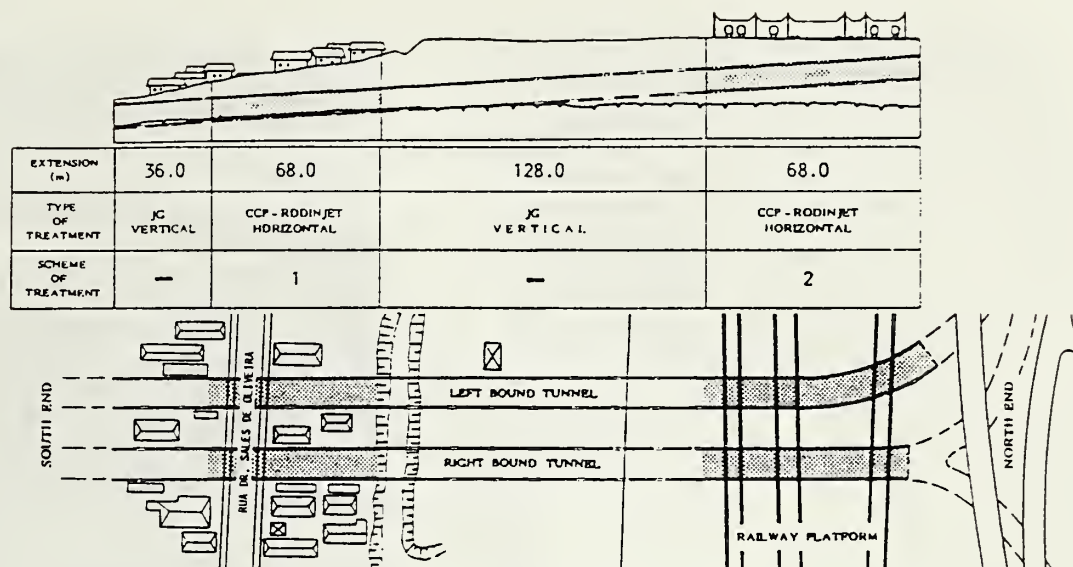


Figure 2. Campinas tunnels a) longitudinal section - b) plan view of twin tunnel

horizontal columns forms a protective shell around the excavation which is then lined with steel ribs and shotcrete reinforced with a steel net. The length of the treatment is normally comprised within 10 and 15 m, the length of excavation ranges between 7 and 12 m. Excavation usually stops 2 or 3 m before the end of treatment in order to leave a sufficient supporting frame of columns into the face of tunnel, to ensure stability.

The consolidation of the soil ahead of the excavation is one of the major advantages of this method, ground movements or decompressions caused by excavation can be prevented or reduced to a minimum. The natural mechanical characteristics and the internal friction in soil, which greatly contribute to the stability around the excavation (creation of a natural arch), can be preserved avoiding large settlements or failures in proximity of the tunnel face area.

The horizontal jet-grouting treatment has been applied for the first time in South America, in Brasil, for the excavation of two urban shallow tunnels of large dimensions in the town of Campinas, 100 km northwest of S. Paulo.

In Fig. 2 a longitudinal section together with a plan view of the two tunnels is shown. This work realizes a bypass between two new expressways under construction, which surround the town center. The two bores underpass an inhabited borough, cross a heavy traffic road and, close to their north end, cross the whole railway platform nearby Campinas central station. The soils interested by the excavation are formed by clastic medium to fine graded sediments. The main representative layers are described on the simplified soil profile along the tunnel axis, shown in Fig. 3. The overall extension of the two tunnels is of 594 m (294 m for tunnel 1 and 300 m for tunnel 2). The excavated section has a polycentric shape 14 m wide (reaches up to 17 m at north entrance of tunnel 1) and 11 m high (5.5 m high at half section excavation), with a total area of 125 sq.m. The jet-grouting treatment has been used on 254 m of tunnel length (40% approx. of total extension). Between September and December 1987, 136 m on both tunnels (68 m each) were excavated with the aid of sub-horizontal treatment, at the south end (see Fig. 2). An important and crowded town road and various inhabited buildings were crossed with an overburden varying between 4 and 7 m. From January 1988 the excavation

from the north side portal (see Fig. 2) started on both tunnels. A total of 118 m of tunnel were excavated under the sub-horizontal treatment (50 m for tunnel 1 and 68 m for tunnel 2) with an overburden varying from a minimum of 3.5 m at the north end up to 5.5 m.

The excavation underpassed several railway tracks of the FEPASA (Ferrovia Paulista S/A) on the S. Paulo-Limeira line which remained under use for the whole duration of the underground works. The rest of the tunnel was treated from surface by means of vertical JUMBO-GROUTING (two ways system) to form 1.5 m thick arches of cemented soil above the tunnel crown. The drilling rig used for sub-horizontal jet-grouting is shown in Fig. 4. It is hydraulic and crawler-mounted. The entire arch, formed by sub-horizontal columns, is obtained using the rig's moving mast positioned at the crown arch path. The mast swings about the crown axis without requiring further rig displacement. At present the maximum drilling length is 10.5 m. In Fig. 5 the two different treatment schemes adopted for the south and north bound tunnels respectively, are reported (in cross and longitudinal section). It can be noticed that a heavier treatment was adopted at the north end, this variation from the initial project was needed because of the following factors: the reduced tunnel cover, which increases the non-homogeneity of loads acting on the arch, the dynamic effects produced by the running trains, the weakness of the porous soft sandy clay layer located at the top of the crown.

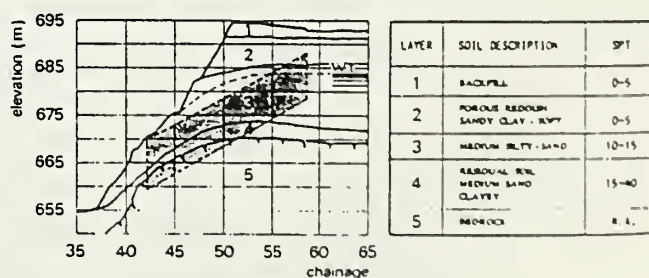


Figure 3. Soil profile along tunnel axis



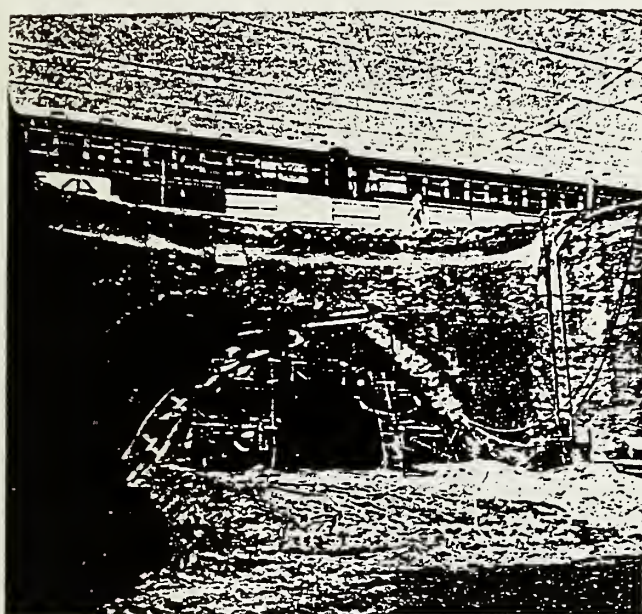
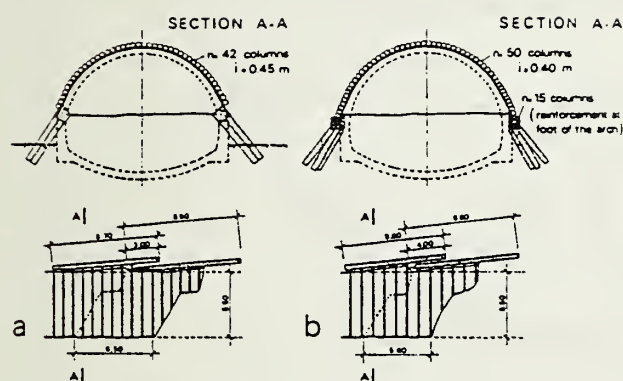


Figure 4. RB-22 Rotary drilling rig

Figure 5. Treatment schemes  
a) scheme n. 1 - south end - b) scheme n. 2 - north end

## INJECTION PARAMETERS AND LABORATORY TESTS

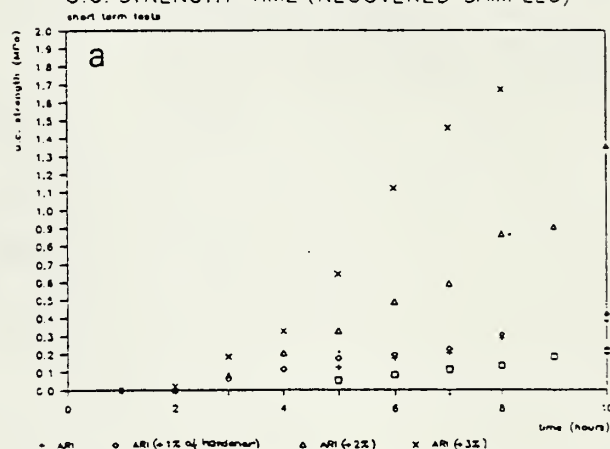
In Fig. 6 the injection parameters and cement grout compositions are shown. ARI type cement was used (performing high resistance and quick hardening) in order to reduce disturbance to adjacent columns during the injection, to minimize settlements caused by jet-grouting and to reach at short term a minimum strength of the arch to allow excavation soon after the end of treatment. During the works some samples of cemented soil were recovered directly from inside the bodies of the columns already injected. The results, in terms of unconfined compression strength of samples versus time are reported on Fig. 7. The two plots show the evolution of resistance at short term (first 10 hours of ageing) and medium term (14 days). Some tests were performed adding a hardener to the grout (with percentages of 1, 2 and 3% on the cement weight). The strengths obtained by the tests were used to decide the type of cement to be adopted and the eventual need of hardeners. In addition some measurements of elastic modulus at 2 and 7 days ageing were performed, using cement ARI with a water/cement ratio of 0.8.

## JET GROUTING PARAMETERS USED FOR SUB-HORIZONTAL TREATMENT

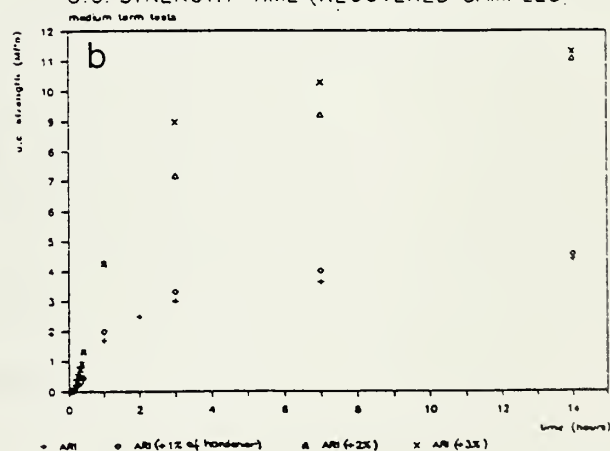
INJECTION PRESSURE	:	35.0 (MPa)
CEMENT TYPE	:	ARI
WATER/CEMENT RATIO	:	0.8
GROUT DENSITY	:	1.59 (kg/dm <sup>3</sup> )
CEMENT CONSUMPTION PER M OF COLUMN	:	250.0 (kg)
GROUT CONSUMPTION PER M OF COLUMN	:	283.0 (l)

Figure 6. Injection parameters summary table

## U.C. STRENGTH-TIME (RECOVERED SAMPLES)



## U.C. STRENGTH-TIME (RECOVERED SAMPLES)

Figure 7. U.C. tests on columns samples  
a) short term tests - b) medium term tests

The average values obtained are reported herebelow:

AGEING (days)	Average Elastic Modulus (MPa)	E/S (*) ratio (-)
2	1600-1800	600-700
7	2200-2500	

(\*) E/S = elastic modulus / u.c. strength ratio



## TOPOGRAPHIC MEASUREMENTS

Measurements of settlements and ribs convergence, if carried out with sufficient continuity and accuracy, together with the progression of works, represent an essential instrument for monitoring the behaviour of the excavation, in order to verify the existence of the conditions of stability and safety and to control the amount of differential and total settlement caused by the excavation in progress and, at last, to perform a back-analysis of the project.

The following measurements were performed:

- settlements of datum points on surface
- deep soil displacements with use of rod extensometers
- settlements of steel ribs at top and foot
- convergence of ribs.

Settlements on surface reached a maximum of 30-60 mm before stabilization. In Fig. 8 settlements at surface versus time are plotted for a representative instrumented section of tunnel 1, together with face advance.

The jet-grouting treatment is responsible for a certain portion of the total settlement (15-20 mm approx.), this effect can be connected with two unfavourable elements; the extremely reduced overburden with respect to the size of the bore and the low consistency of the porous soil.

All settlement-time curves show a good stabilization at a distance of two to three diameters behind the face.

The excavation of the bottom part of the tunnel caused only small additional settlements. The measures of convergence performed during the first step excavation show reduced values everywhere, with strains in the range of  $0.5-1 \times 10^{-3}$ . A larger increase of convergence was observed during the excavation of the invert, the final values reached  $2.5-3 \times 10^{-3}$ .

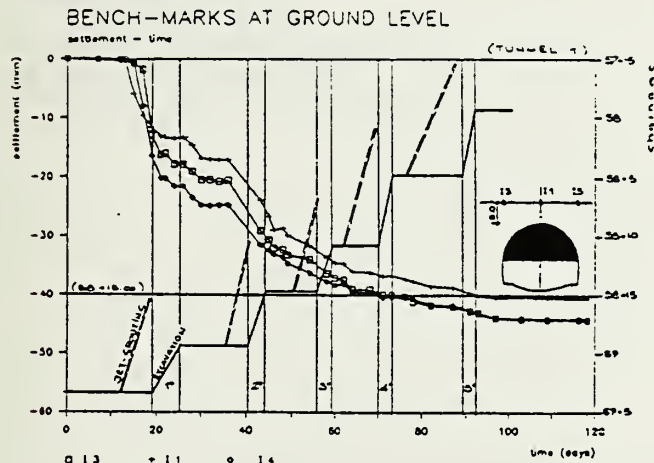


Figure 8. Surface settlements versus time (monitored section at chainage 58 + 15 m - tunnel 1)

## CURRENT DEVELOPMENTS IN BRASIL

The successful work carried out at the Campinas tunnel proved the effectiveness and affidability of sub-horizontal jet-grouting treatment applied to the Brazilian soils. At present another tunnel in the town of S. Paulo is under excavation with the aid of this method.

It is a twin tunnel, part of a project called "mini expressway ring" surrounding the town center. The two tunnels have the same dimensions of the Campinas ones, with an overall length of 750 m each. The use of this soil improvement method is foreseen in other urban tunnels in project and under construction at present in S. Paulo, as the twin tunnels undergoing the Ibirapuera park (late 1988).

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## Horizontal jet grouting as a temporary support for the "Monteolimpino 2" tunnel

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**ABSTRACT:** Two portions of the "Monteolimpino 2" tunnel for a total of 1000 m approx. was located in alluvial silty sands and gravels. The 12 m wide tunnel was advanced rapidly through the granular deposits by forming in radial planes ahead of the excavation face an arch support of slightly outward inclined and overlapping jetgrouted soil-cement columns. Measurements of settlement and horizontal displacements at the surface as well as determination of strain distribution along vertical boreholes in two different sections demonstrated that the construction method adopted caused small ground movements without appreciable variation of the state of stress.

## 1. INTRODUCTION

The Italian section of the Gotthard railway line connecting Milano to Chiasso and Basel (Switzerland) even though more than a century old still maintains characteristics of efficiency consistent with its huge volume of traffic: 130 trains a day amounting to a yearly traffic circulation of 3.2 million of passengers and 8 million tons of goods.

But there is a portion between Chiasso and Albate Camerlata, where the too small width of Monteolimpino 1 tunnel does not permit the transit of the modern containers. Besides the heavy slope (1.8%) limits the speed of the trains so much that, in some cases, an additional locomotive is required.

To solve these inconveniences the construction of a line alternative to the existing one was decided.

The principal structure of the alternative railway section is the Monteolimpino 2 tunnel. This is 7209 m long and has a net radius of 5 m in accordance with European Railway Standards (Fig. 6).

## 2. GEOLOGY

Starting from the southern adit of Albate, the Monteolimpino tunnel (fig. 1) crosses from South to North the following formations:

- alluvial and glacial deposits for 250 m

approx.

- a sandy-marl formation for 1500 m approx.
- alluvial deposits for 750 m near the Grandate plain
- 4500 m of sound rock consisting first of conglomerate and then of limestone
- finally 200 m glacial deposit.

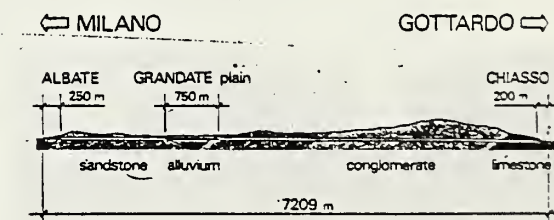


Figure 1. Section of tunnel along axis showing crossed geological formations.

## 3. WORKSITE ORGANIZATION

In order to respect the 56 months working schedule, the tunnel construction was organized so as possible inconveniences met during excavation through glacial and alluvial deposits would not interfere with the general working program.

To this aim, in the Southern side of Albate, a drift was excavated in order to by-pass the initial stretch of loose ground.



Northwards side Chiasso, the tunnel adit was obtained by cutting the slope with a diaphragm wall and a 3.60 m diameter pilot drift was then bored southwards up to Grandate plain by a TBM.

In the Grandate plain, an access shaft was excavated with the aid of reinforced concrete diaphragm walls, down to 35 m of depth. This allowed to attack on two fronts the most difficult tunnel portion in loose sand with very low water content.

From the construction point of view, the rock sections of the tunnel even though representing the six sevenths of the route, did not involved particular difficulties. The tunnel was excavated full section by blasting with an average rate of 6 m per working day for the stretch southwards the Grandate plain, and of 12 m/day for the Northern one, thanks to the presence of the pilot drift and to the soundness of the rock.

On the contrary the difficulties met during the excavation of the access drifts led to the decision of adopting a soil consolidation treatment ahead of the excavation in loose ground (i.e. in the Grandate plain and in the first alluvial stretch of Albate).

#### 4. PRELIMINARY SOIL CONSOLIDATION BY JET-GROUTING

Usually a preliminary soil improvement for tunnelling in soft or loose ground is aimed to:

- convey the deviated stresses along paths formed by improved soil
- limit deformations
- attain a regular excavation section (no asymmetrical loading)
- reduce loosening pressures providing a prompt radial support.

This is obtained by favouring the formation of a sort of transversal arch effect. Should this be achieved before the excavation (i.e. at a distance, ahead of the excavation face, as long as one tunnel diameter approx.) the tendency of the face to be extruded highly decreases and its three-dimensional contribution to the opening stability is emphasized.

In the case of the Monteolimpino 2 tunnel, among other solutions like traditional grouting or soil freezing, jet grouting technique was chosen to save both money and time.

Compared with traditional (permeation) grouting, jet grouting avoids penetrability problems in connection with grain size distribution of silty sands and those relevant to durability and possible pollution of chemical products.

Jet grouting consists in fracturing and simultaneously mixing the soil in situ with a cement grout injected through small radial nozzles under very high pressures (up to 50 MPa).

The operation sequences can be summarized as follows:

- drilling down to the required depth by use of a string of rods fitted at the bottom with a drilling and jetting tool
- grout jetting while revolving and drawing up the drill string.

The diameter and the mechanical characteristics of the columns depend both on the soil features and the combined effects of the construction parameters, i.e.: grout composition, flow rate and pressure, number and diameter of the nozzles, rotational and withdrawal speed.



Figure 2. Jet grouted columns recovered from the tunnel face.

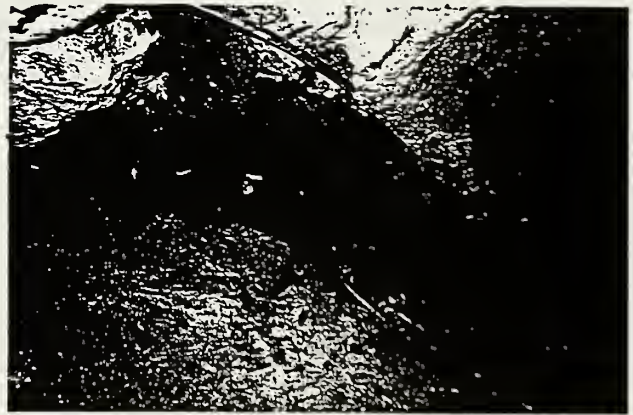


Figure 3. Detail of jet grouting treatment along the contour of the excavation face.

In granular soils the single column diameter usually ranges between 50 and 70 cm (fig. 2) and a consolidated soil arch can be formed by partially overlapping adjacent columns carried out along the contour of the excavation face (fig. 3).



The unconfined compression strength of the jet grouted soil is of the order of 15 MPa.

Although vertical jet grouting had been developed in Japan more than 20 years ago and has been known in Europe for more than ten years, the use of jet grouting executed horizontally is relatively recent.

This application, promoted by P. Lunardi could be practically applied when Rodio Co. decided to finance a trial field which proved the feasibility of horizontal jet grouted columns (Tornaghi R., Perelli Cip-po A., 1985).

The second step was the construction of a special rig that made possible the execution of columns suitable for the treatment of tunnels even of large diameter (12 to 13 m), providing a reasonable rate of tunnel progress.

The first application on a real scale was carried out in winter 1983-84 on a section of the double-track Campiolo railway tunnel on the Udine-Tarvisio line in northern Italy. Fig. 4 shows the specially designed drilling rig SR 500: a system of hydraulic jacks allows the mast to be rotated 180° with adjustable inclination as to the horizontal.

A year later, the treatment of Montelimpino 2 tunnel was started. In this job the prototype of the second generation of rigs was employed: the SR 510 rotary percussion drilling rig (fig. 5). This rig is equipped with a movable platform consenting a more direct access of the operator to the face and a greater versatility in respect to treatment geometry.

On account of the many successful achievements this technique has developed with prodigious celerity first in Italy, where at least 7 types of special rigs are now on the market, and then in Austria and in Germany.

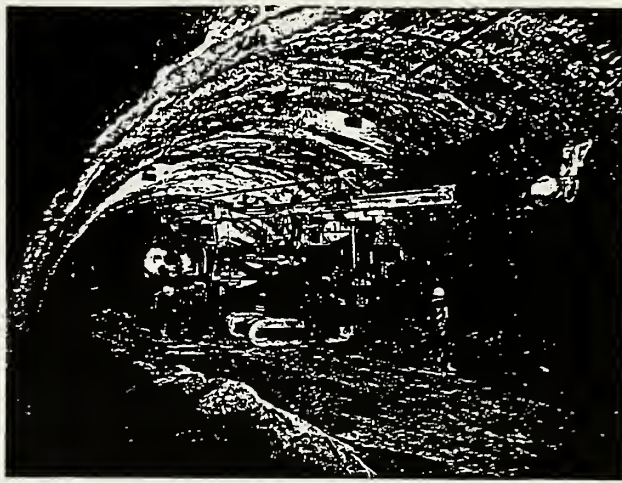


Figure 5. SR 510 drilling rig.

## 5. CONSTRUCTION STEPS

In fig. 6 the tunnel construction steps in the sandy soils of the Grandate plain are summarized (for soil features see para 6.1.):

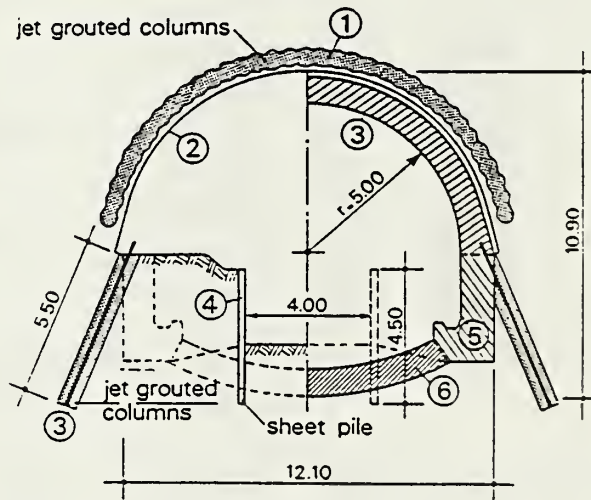


Figure 6. Tunnel construction steps in alluvial soil.



Figure 4. SR 500 drillig rig.

1. Formation of the consolidated soil arch consisting of 35 jet grouted columns with 0.45 spacing and 11° outward slope to the tunnel axis. Fig. 7 shows the columns geometry based on holes 10 m long of which 7 m grouted and 3 m simply plugged. In this way 1 m overlapping between two subsequent treatments is provided. Each treatment permits the excavation of the top section of the tunnel for a length of 6 m, still maintaining the treated arch 4 m ahead of the face.

The geometry and length of the columns were so calibrated as to adjust the jet



6. Inverted arch construction. After the sheet-piling was withdrawn the inverted arch was excavated and concreted.

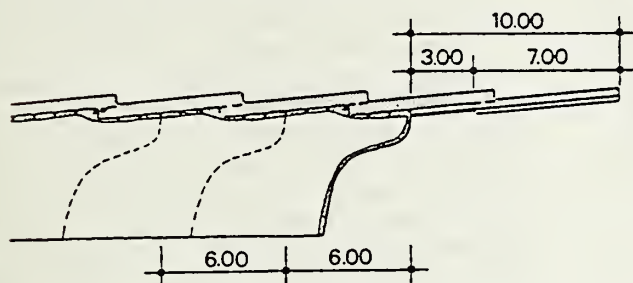


Figure 7. Longitudinal configuration of A 6.1. Site Investigation jet grouted columns.

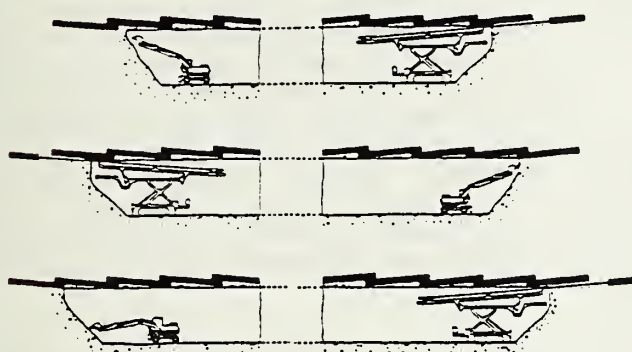


Figure 8. Alternating jet grouting and excavation phases.

4. Placing of vertical sheet-piling and excavation of the soil between them. Each sheet piling row was interrupted every 9 m to allow the subsequent construction of

## 6. MONITORING PROGRAM

To verify the jet grouting treatment effectiveness on the stabilization of the soil surrounding the opening, two sections were instrumented and a site investigation program was conducted.

## 6.1. Site Investigation

The following tests and determinations were made:

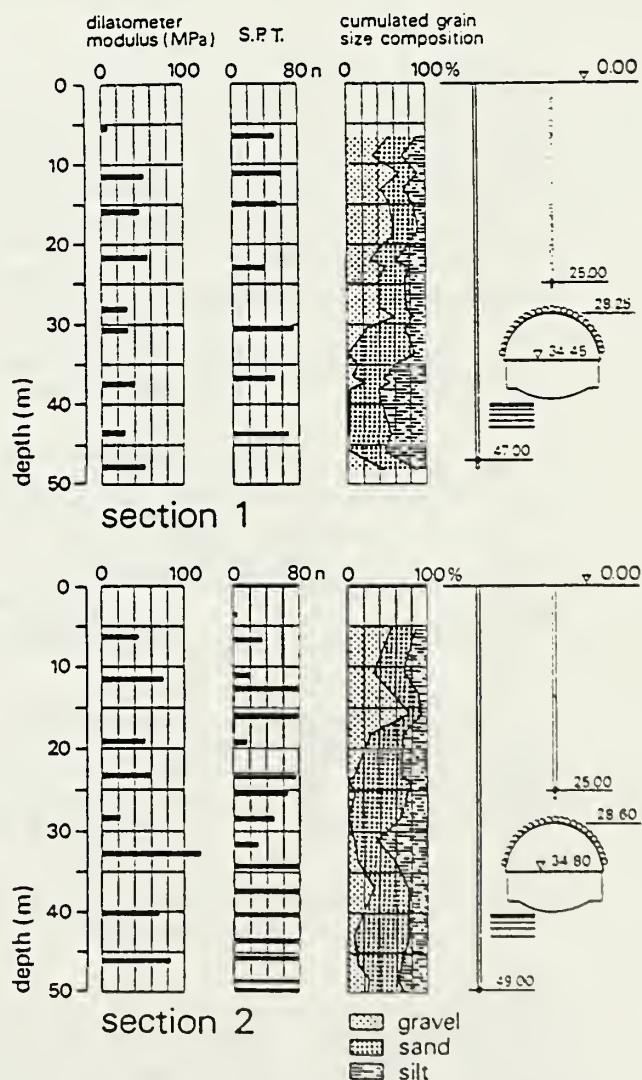


Figure 9. Soil profile of the instrumented sections.



- cored boreholes, 50 m deep
- Standard Penetration Tests (SPT)
- determination of borehole-alignment by Eastman Multishot
- grain size distribution, water contents
- determination of elasticity modulus E by in situ dilatometer tests (Thut, 1977).

The main results are shown in fig. 9. It can be seen that around the tunnel the soil modulus of elasticity in section 1 is significantly lower than in section 2. This is confirmed by SPT values and grain size distribution. The average water content ranges between 6 and 18 per cent as a function of the clayey silty content. The low plasticity of fines made the determination of Atterberg's limits questionable.

## 6.2. Instrumentation

Nine topographic survey monuments were placed at ground surface along a cross section, according to scheme of fig. 10.

Two sliding micrometers were installed in vertical holes. The first one at the centerline of the tunnel, the second 10 m from the tunnel axis. Sliding micrometer (Kovari, Amstad Ch. 1983) consists of a high precision probe which is inserted in a special casing to measuring the relative

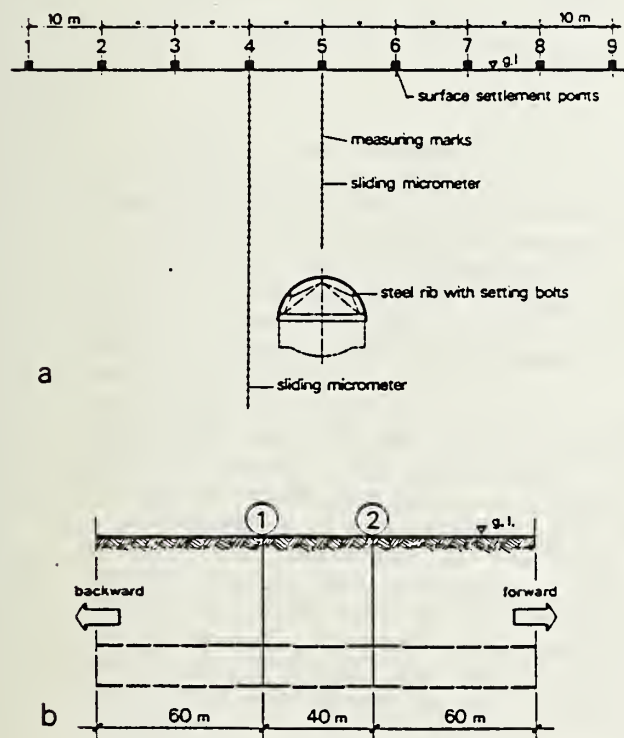


Figure 10. Arranging of measuring (a) and relative location (b) of instrumented cross sections.

distance of consecutive couplings spaced one meter apart. By repeating the measurements at given time intervals the axial strain evolution of the surrounding soil to which the casing is bound, can be obtained.

Readings of all the instruments were taken during the various working phases:

- the excavation of the upper part was followed over 160 m long stretch of tunnel, from 60 m before section 1 to 60 m after section 2 (fig. 10 b);
- the other excavation phases were covered within a measuring period of 21 months.

## 6.3. Surface Settlements

The geodetically measured surface settlements of section 1 and 2 are illustrated in fig. 11 as a function of face position during crown excavation. The values of settlements can be summarized as follows:

	section 1		section 2	
point	1.5	1.4	2.5	2.4
total settlement (mm)	30.1	20.5	21.5	15.5
settlement at break through (mm)	8.5	5.1	5.5	3.2
percentage of break through to total settlement	27	25	26	20

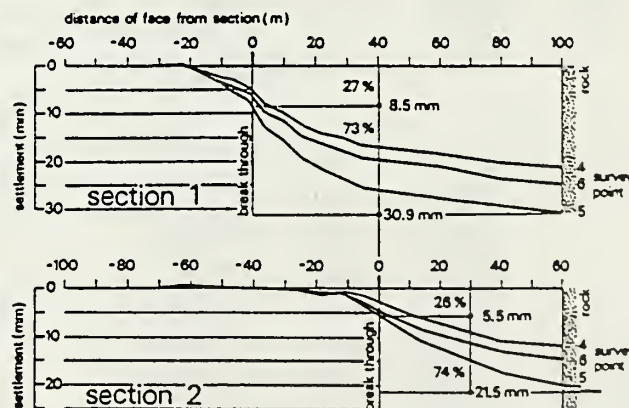


Figure 11. Surface settlements as a function of face position.

Assuming the bottom of the 50 m long tubes as a fixed point, the integration of the sliding micrometers results along lines 1.4 and 2.4 (fig. 14) give settlement values of 22.9 and 15.7 mm which are very closed to those geodetically measured.



grouting construction times to those of the excavation. In this way the two crews for injection and excavation respectively could be alternatively moved on two opposite fronts thereby avoiding work interruptions and idle times (fig. 8).

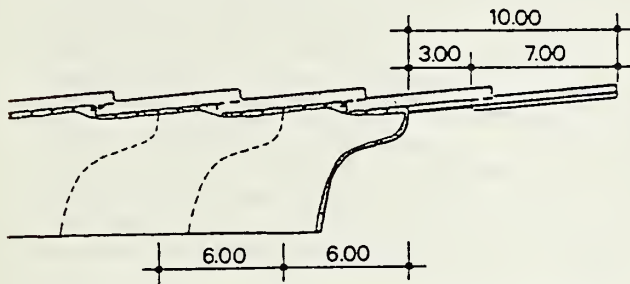


Figure 7. Longitudinal configuration of A jet grouted columns.

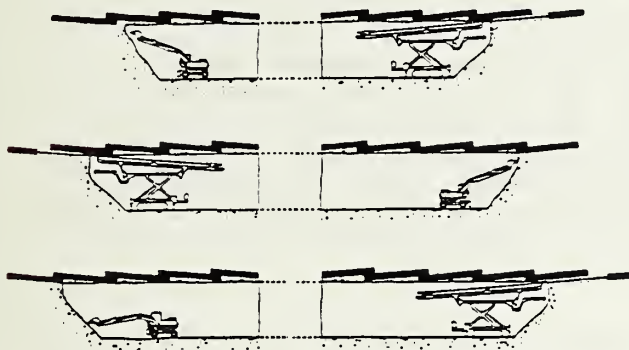


Figure 8. Alternating jet grouting and excavation phases.

2. Tunnel top section excavation and prelining. This consists of 1 m spaced steel ribs and 15 cm thick shotcrete.

According to this working sequence, the tunnel top section was carried out at a rate of 2.4 m approx. per working day. Generally it was possible to complete a single jet grouted arch in 19 hours using a crew of 4-5 workers.

An average of 0.2 m<sup>3</sup>/m of grout was required for each column. The C/W ratio normally was about 1.25 (this means a take of 170 kg of cement per metre of column).

3. Execution of sub-vertical jet grouted columns and concreting of the crown permanent lining. Two retaining walls were formed by reinforced jet grouted columns 0.60 m spaced. Their function was to support earth pressure and permanent lining during the subsequent phase of bottom excavation.

4. Placing of vertical sheet-piling and excavation of the soil between them. Each sheet piling row was interrupted every 9 m to allow the subsequent construction of

the first pier portions in correspondence with the joints of the crown lining.

5. Pier construction on alternate sides of the tunnel in 3 m long portions.

6. Inverted arch construction. After the sheet-piling was withdrawn the inverted arch was excavated and concreted.

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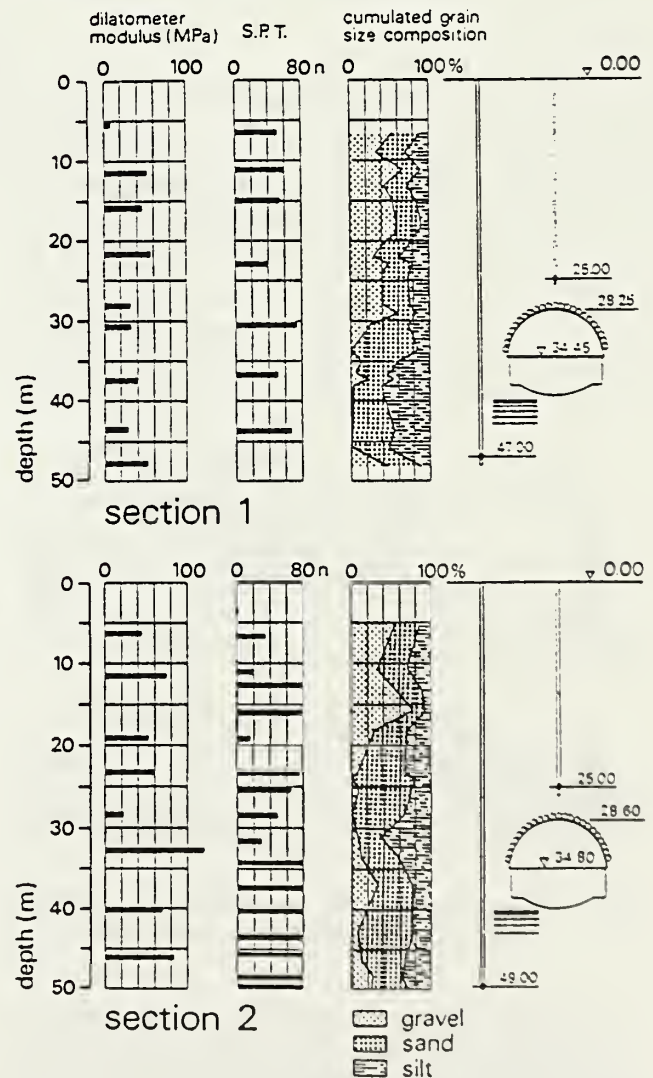


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Nine topographic survey monuments were placed at ground surface along a cross section, according to scheme of fig. 10.

Two sliding micrometers were installed in vertical holes. The first one at the centerline of the tunnel, the second 10 m from the tunnel axis. Sliding micrometer (Kovari, Amstad Ch. 1983) consists of a high precision probe which is inserted in a special casing to measuring the relative

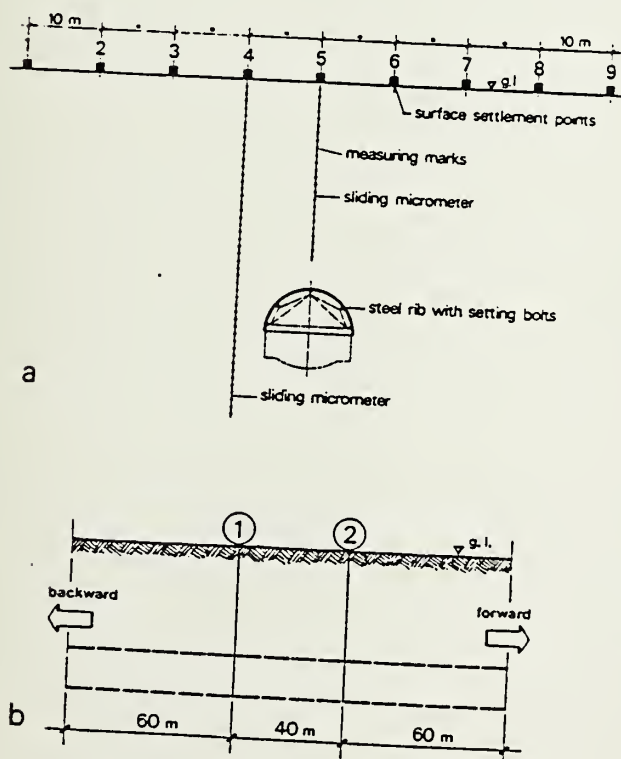


Figure 10. Arranging of measuring (a) and relative location (b) of instrumented cross sections.

distance of consecutive couplings spaced one meter apart. By repeating the measurements at given time intervals the axial strain evolution of the surrounding soil to which the casing is bound, can be obtained.

Readings of all the instruments were taken during the various working phases:

- the excavation of the upper part was followed over 160 m long stretch of tunnel, from 60 m before section 1 to 60 m after section 2 (fig. 10 b);
- the other excavation phases were covered within a measuring period of 21 months.

## 6.3. Surface Settlements

The geodetically measured surface settlements of section 1 and 2 are illustrated in fig. 11 as a function of face position during crown excavation. The values of settlements can be summarized as follows:

	section 1		section 2	
point	1.5	1.4	2.5	2.4
total settlement (mm)	30.1	20.5	21.5	15.5
settlement at break through (mm)	8.5	5.1	5.5	3.2
percentage of break through to total settlement	27	25	25	20

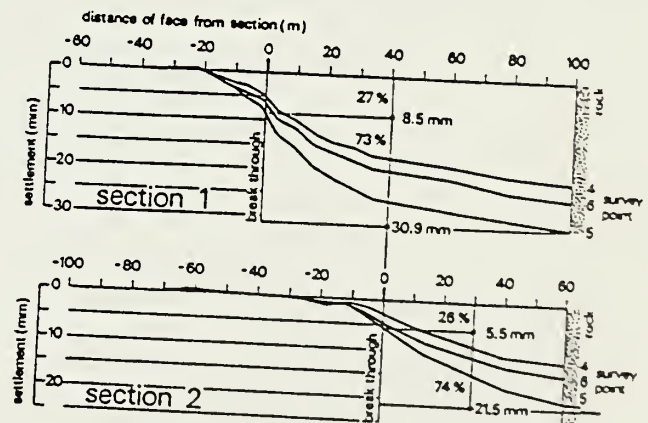


Figure 11. Surface settlements as a function of face position.

Assuming the bottom of the 50 m long tubes as a fixed point, the integration of the sliding micrometers results along lines 1.4 and 2.4 (fig. 14) give settlement values of 22.9 and 15.7 mm which are very closed to those geodetically measured.







The tunnelling longitudinal effect is evidenced by the following:

- a) a slight swelling occurs more than two tunnel diameters ahead of the excavation face;
- b) when face crosses the observed section the settlements are in the range of 26 to 27 per cent of the total settlement;
- c) the settlements tend to stop when the face reaches a distance of about 40 m (3 diameters approx.).

The geodetically measured settlements observed throughout the construction time are illustrated in fig. 12.

Also in this case we can distinguish three zones characterized by different deformation rate:

- 0.3-0.6 mm/day during crown excavation from -20 to +35-40 m of distance of the face from the observed section
- 0.09-0.12 mm/day during excavation of the bottom part, up to piers construction (phase 5)
- 0.01-0.03 mm/day between piers and invert arch construction (phase 6).

For each section the figure 13 shows the settlement transversal trough, according to the Peck-Attewell scheme.

An estimation of the volume of settlement trough ( $V_s$ ) per unit distance of tunnel progress (= loss of ground volume) and its percentage as referred to the tunnel cross sectional area ( $V_e$ ) is given below:

section	settlement volume $V_s$ ( $m^3/a$ )		relative ground loss $\frac{V_s}{V_e}$ %	
	1	2	1	2
after crown excavation and prelining	1.15	0.83	1.91	1.38
after invert arch construction	2.20	1.22	1.97	1.10

So small values of ground loss have corroborated the suitability of the construction procedures considering that granular cohesionless soils are prone to collapse at the tunnel face creating large vertical deformations.

The final settlements are 58.2 and 38.2 mm for section 1 and 2 respectively whereas the settlements occurred during crown construction represent the 53 and 55 per cent of the total one.

The settlement values observed in section 2 are lower than in section 1. This can be explained having in mind the discrepancies in soil conditions illustrated in chapter 6.1.

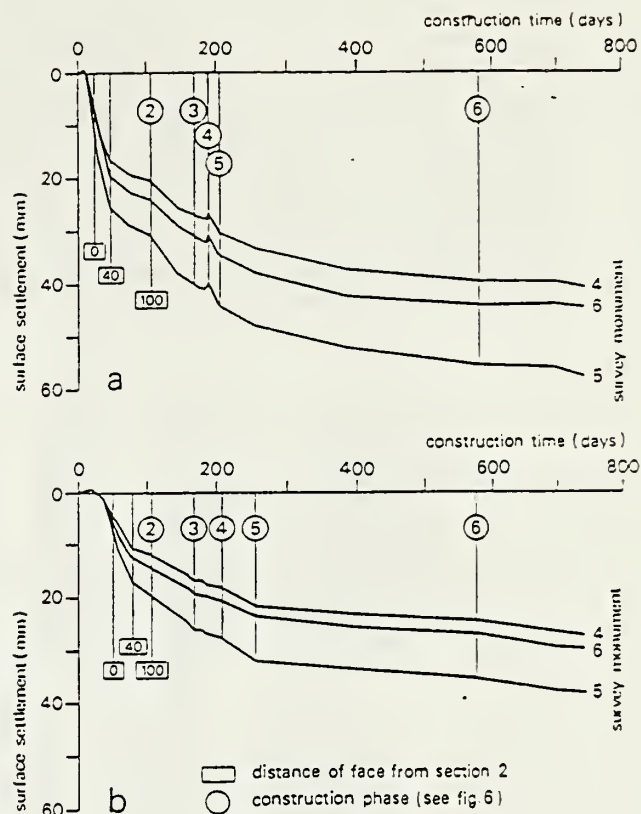


Figure 12. Surface settlements throughout construction time of section 1 (a) and section 2 (b).

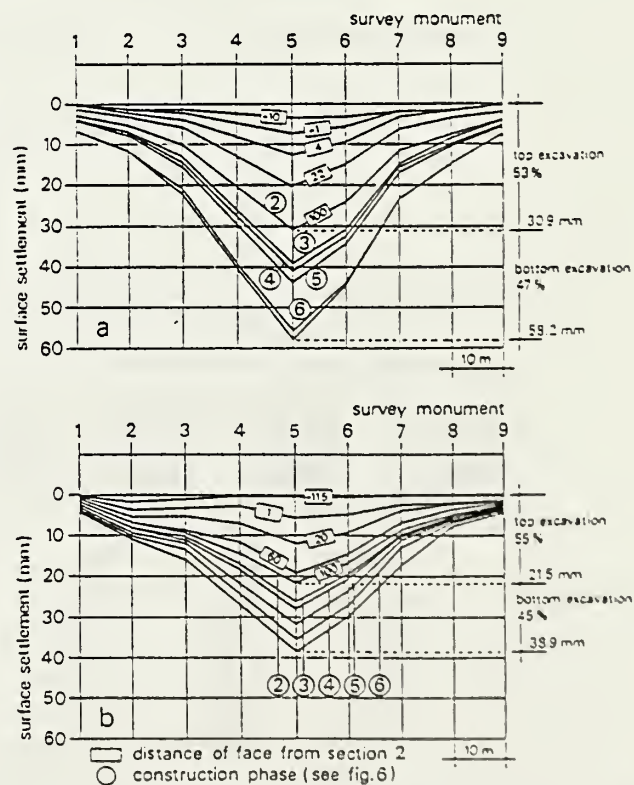


Figure 13. Surface settlement contour at various working phases of section 1 (a) and section 2 (b).



#### 6.4. Soil strains versus depth

Of particular interest to evaluate the effectiveness of jet grouting treatments are the results of the measurements of the Sliding Micrometer, especially those of section 1 as shown in terms of differential and integrated values in fig. 14 and 15.

The line 1.5 (fig. 14), after the breakthrough of the excavation face, shows pro-

longation strains. However, the measured values just above the tunnel vault are lower than the expected ones. This can be understood analyzing the changes of deformations while the tunnel's excavation approaches and crosses section 1 having in mind the correlation between the measurement number and the working sequences detailed in fig. 16. First, compression strain increases up to the measure n. 6 when the jet grouting treatment just above

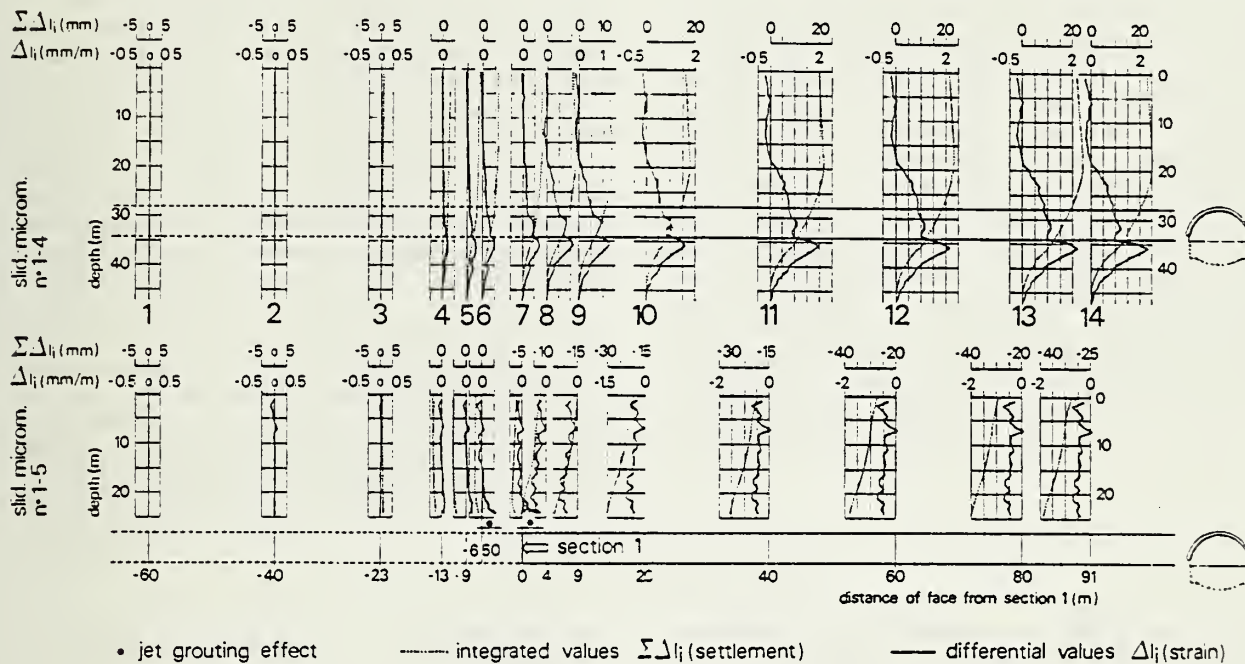


Figure 14. Distribution of strain and vertical settlement measured by sliding micrometer during top portion excavation of section 1.

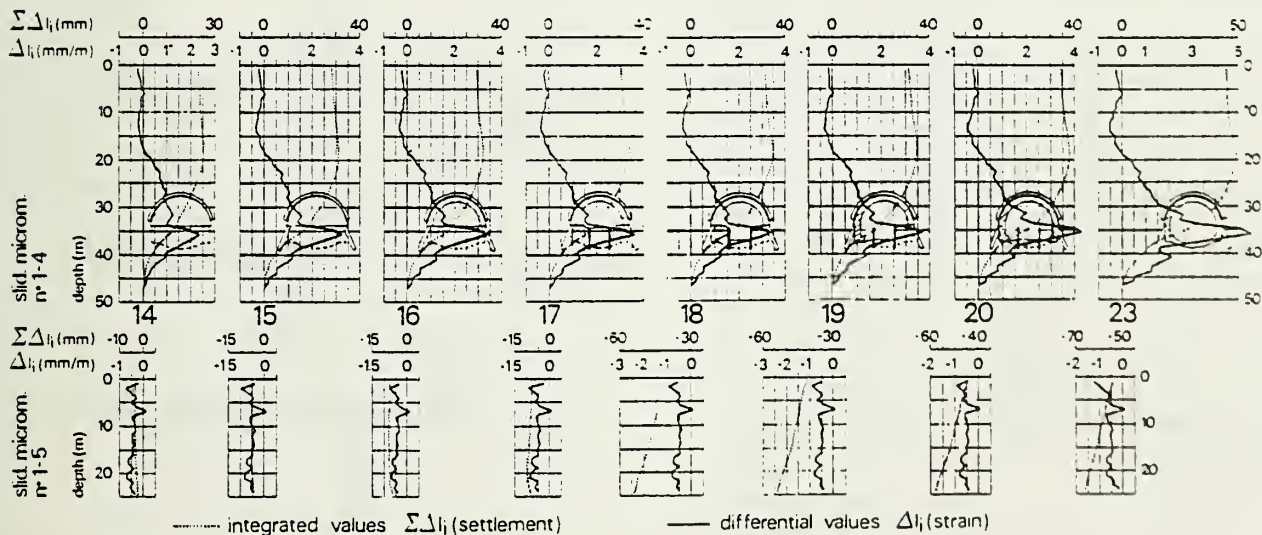


Figure 15. Distribution of strain and vertical settlement measured by sliding micrometer during bottom portion excavation of section 1.



the section 1 is performed. The same type of strain persists also in the measure n. 7 when the excavation face crosses the observed section. Then the strain changes rapidly in prolongation as the excavation proceeds. In our opinion this is the effect of the jet grouting columns construction. Due to an injection pressure of 40 MPa a sort of prestressed arch is formed along the tunnel contour.

The main feature of line 1.4 is the peak of compression strain which characterizes the zone where the jet grouting treatment, intended as a transversal arch, is founded. This peak reaches in the measure n. 14 (when the excavation face is 91 m beyond section 1) the value of 2.3 mm/m. As a consequence more than a half of the total settlement is found below the excavation platform (i.e. below the steel ribs toe).

From the foregoing two considerations may be drawn:

- a) the transverse effect of the jet grouting treatment is well demonstrated
- b) the foundation of the treated soil arch represents a not negligible aspect of the design: in fact quite a number of subsequent applications showed some inconveniences owing to insufficient soil bearing capacity.

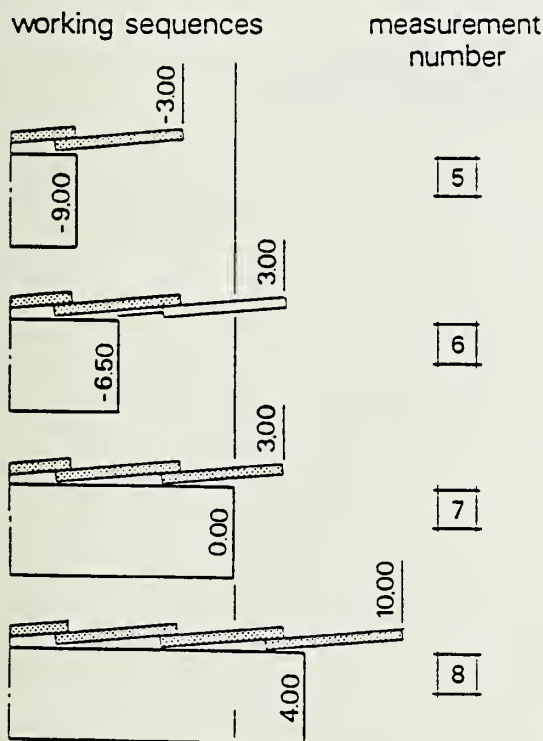


Figure 16. Correlation between sliding micrometer measurement number and working sequences of section 1.

## 7. CONCLUSIONS

Tunnelling through cohesionless silts, sands and gravels can be carried out only by complete protection of the top, sides and face of the excavation, as by full forepoling and breasting, or by rendering the materials cohesive by injection of grout. This statement made by Ralph B. Peck was fully corroborated during the construction of the Monteolimpino 2 tunnel sections in alluvial soil where the horizontal jet grouting technique has been one of the first applications. The results obtained from instrumented sections have demonstrated that this type of treatment works better crosswise than lengthwise. Consequently great care must be given to the stability of the grouted soil foundation especially during piers construction. Surface settlements resulted limited but not negligible, its reduction might have been obtained, if required, by increasing the front support (for instance with a few columns in the centre of the section) and by shortening the sequences of the various construction phases.

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**APPENDIX D**

**REPORT BY WERNER AND ASSOCIATES:  
“UNDERPINNING OF EXISTING HISTORIC BUILDINGS”**



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CALTRAIN S.F. DOWNTOWN EXTENSION  
SAN FRANCISCO CALIFORNIA

## UNDERPINNING OF EXISTING HISTORIC BUILDINGS

REPORT PREPARED  
BY  
WERNER & ASSOCIATES  
FOR DAMES & MOORE



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MAY 1996





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May 5, 1996  
W & A. Job No. 95084

Dames & Moore  
221 Main Street, #600  
San Francisco, CA 94105

Attn: Mr. Demetrious Koutsoftas,  
Principal

Gentlemen:

RE: Report on Underpinning of Existing Historic Buildings,  
Caltrain S.F. Downtown Extension  
San Francisco, CA

## 1. INTRODUCTION

This report summarizes the findings of a feasibility study on support and underpinning of existing buildings which may be affected by the large railroad tunnel excavation for the above-referenced project.

The existing historic buildings which are under consideration in this report are all located within an area bounded by Townsend, 3<sup>rd</sup>, Colin P. Kelly, Jr. and Brannan Streets.

Presently, there are three alternate tunnel alignments under consideration for this area. The basic difference between the three proposed routes is the average radius of curvature for the 90-degree turn-up from Townsend Street and onto the Colin P. Kelly, Jr. Street alignment.

One option depicts a long radius tunnel, another a medium radius, and finally, the third option is a proposal for a short radius tunnel.

Accordingly, since the long radius route makes the turn gradually over a substantial diagonal, the number of buildings under which the railroad tunnel is driven for this alternate is far greater than for the short radius route, which follows the actual street corner more closely; i.e., the distance that the long radius route passes directly underneath existing buildings is longer than for the short radius route (see pages three & four for a summary).

In this report, we have therefore elected to refer to these three alternate alignments as the long, medium and short radius tunnel routes, since both the length of tunnel under existing buildings as well as the average radius curve description of each route are proportional.



However, it should also be pointed out that the long radius route is in fact the shortest tunnel distance between the Townsend & 3<sup>rd</sup> and Colin P. Kelly Jr. & Brannan Street intersections. Conversely, the so-called short radius route is actually the longest tunnel in overall length.

During tunnel excavation, there exists the potential for deformation of the surrounding soil/rock mass. Several support methods can be implemented in order to reduce or eliminate the likelihood of settlement to buildings founded directly above the actual tunnel alignment. One such option is underpinning of the foundations of affected structures.

Underpinning by definition means to extend an existing footing by various methods to bear on material located below the influence of such adjacent and deeper excavation.

Other options exist; e.g., improved and conservative tunneling methods, ground modification (strengthening) methods, segmented tunnel driving, etc. These other options are being evaluated by others.

## **2. PURPOSE AND SCOPE**

The purpose of this investigation is to define and review various alternative approaches to the underpinning and present the advantages and disadvantages of each of the methods. It also includes the presentation of cost estimates for the alternate underpinning schemes considered for each of the three railroad tunnel alignments.

The scope of our services includes, but is not necessarily limited to, the following tasks:

- 2.1 Review of existing geotechnical information, building data, alternate tunnel alignments, and performance criteria.
- 2.2 Review of building construction and foundations, site information, and tunnel lining methods.
- 2.3 Participation in meetings to explore the various options being considered.
- 2.4 Develop cost models for the underpinning work.
- 2.5 Preparation of a final report summarizing the findings and recommendations.



Specifically, this report is based on our review of the following:

- a) Available soils information, including boring logs, profiles, and test results prepared by Dames & Moore.
- b) Field sketches and copies of existing structure plans produced by AGS, Inc.
- c) Walking tour and site reconnaissance of existing historical buildings performed by this writer.
- d) Alternate tunnel alignments developed by ICF Kaiser Engineers (ICFK) and DeLeuw Cather & Company (DCCO).
- e) Alternate tunnel methods and ground stabilization schemes proposed by Haley & Aldrich (H & A) and Nicholson Construction Company (NCC).
- f) General information gathered from various meetings on this topic.

### **3. TECHNICAL APPROACH**

We have studied the proposed tunnel alignments and we have developed the following list of "affected" buildings for each of the three alternative tunnel routes:

#### **SHORT RADIUS TUNNEL**

- 1. 625 2<sup>nd</sup> Street (nominally)
- 2. 699 2<sup>nd</sup> Street/96 Townsend Street
- 3. 62 Colin P. Kelly Street
- 4. 52 Colin P. Kelly Street

#### **MEDIUM RADIUS TUNNEL**

- 1. 136 Townsend Street
- 2. 130 Townsend Street
- 3. 698 2<sup>nd</sup> Street
- 4. 680 2<sup>nd</sup> Street
- 5. 670 2<sup>nd</sup> Street
- 6. 625 2<sup>nd</sup> Street
- 7. 275 Brannan Street



### **LONG RADIUS TUNNEL**

1. 180 Townsend Street (nominally)
2. 178 Townsend Street
3. 162 Townsend Street
4. 148 Townsend Street
5. 144 Townsend Street
6. 136 Townsend Street
7. 130 Townsend Street
8. 75 Stanford Street
9. 650 2<sup>nd</sup> Street/55 Stanford Street
10. 640 2<sup>nd</sup> Street
11. 625 2<sup>nd</sup> Street
12. 611 2<sup>nd</sup> Street
13. 275 Brannan Street

Obviously, from an underpinning standpoint, the short radius tunnel route is the most favorable.

In this section (technical approach), we developed three "underpinning" options. For each option, the advantages and disadvantages are discussed in general terms. Each method is also described in detail by a narrative and attached sketches, as they apply.

The "zones of influence and of no-load", which sets the limits of underpinning and where underpinning should not be founded, respectively, is also described in this Section 3.

#### **3.1 NO UNDERPINNING**

Concentrate the design effort on developing conservative and improved tunneling methods as a mean to avoid underpinning altogether.

In this manner the buildings are left unsupported and allowed to settle a nominal amount. After tunnel is completed, the cosmetic damage is repaired and building restored, as required.

The more common and conventional underpinning schemes such as deep-mined pits/piers or jacked pipe or pin piles would be difficult, risky, and very expensive to install under the



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Page Five

existing historical buildings. In our opinion, any attempt at performing such type of underpinning construction could increase the potential risk of damaging these structures.

Deep excavations for piers would be difficult in the rock materials and would have to be carried to such depth that they would not be cost effective. Also, such piers, in order to stay clear of the tunneling zone, would have to be spaced so far apart that it would require very large and heavy support beams to span such a distance between piers.

Jacked piles cannot be installed into the hard materials because the existing buildings lack the dead weight necessary to create a reaction to such operation.

However, drilled piles could be a viable option, as will be discussed below.

Obviously, this method of "no underpinning" would then rely heavily on the complete adequacy of the tunneling method to properly support adjacent ground.

Hence, in the event of a catastrophic cave-in during tunnel excavation, the affected building could collapse. It is also quite possible that in such an event, even buildings "protected" by the other two schemes would sustain serious damage, including partial to total collapse.

The point, we want to make here, is that no matter how much a building is underpinned and supported, there still cannot be any guarantee that the building would be unaffected by a major cave-in of tunnel heading.

Therefore, we believe that a very viable strategy for this project would be to concentrate on the development of conservative tunneling methods, in combination with proven and effective schemes for stabilizing the adjacent ground/rock mass from within the tunnel excavation.

In this manner, we believe the differential settlement of these sensitive and fragile historical buildings can be kept to an acceptable minimum.

Nevertheless, in the event that some form of building protection (besides making the tunneling operation safer) will be required, we recommend control underpinning as the preferred option. For description, see Section 3.2 below.

A significant advantage of this scheme is that it will result in far less intrusion to occupied space in the buildings than the third method described in Section 3.3 below, and it is also very cost effective.



However, control underpinning is not as positive as regular underpinning, and may lead to extensive cosmetic damage if the rate of ground settlement is greater than what the maintenance crews, jacking up buildings, can keep up with.

### **3.2 CONTROL UNDERPINNING**

This is actually not underpinning at all, but a method whereby the existing building is being carefully monitored for deformation and then corrected for such movement on a continuous basis.

Accordingly, the ground adjacent to or below the building is allowed to settle a nominal amount while the building is maintained in its original position.

However, this method also allows for the jacking load to be introduced to existing column and wall footings in advance of tunneling operation below.

This will accomplish a form of preloading, which will minimize actual settlement and would allow for adjustments in some buildings that have already experienced differential settlements prior to this work.

#### **3.2.1 EXTERIOR WALLS AND FOOTINGS**

Small and shallow pits are excavated under perimeter footings at distinct spacings, then filled with concrete to form a pier.

Hydraulic jacks are placed in pier pockets under existing footings. In this manner, the building can be jacked up as required to compensate for the possible footing settlement caused by the adjacent tunnel excavation. At completion of work, the space between pier and existing footing is grouted solid after jacks are removed. This method is illustrated in plate SK-1.

#### **3.2.2 INTERIOR COLUMN FOOTINGS**

The principle for control of underpinning at individual column footings is similar to that of the continuous wall footing, except no new jacking piers need to be installed. The existing footing is used as a lower jacking reaction and is permitted nominal settlement, while the top reaction is provided by attached support brackets directly to column above. Jacks are maintained in a ready state between column brackets and top of existing footing. This method is illustrated in plate SK-2.



Obviously, for this mechanism to work, the footing must be first be disconnected from the column. Wood and steel columns are readily disconnected by either loosening of attachment brackets or anchor bolts. For concrete columns, we propose that the perimeter reinforcement be exposed, made ready and to be cut; but only if and when necessary, should movements actually occur.

At completion of work as stated above, any space created from control jacking is grouted and columns re-attached as required, and cosmetic damage repaired.

Piers used in Scheme-I are commonly referred to as "control piers", and the Scheme-II procedure has been called "column pick-up".

These two systems were installed in structures along Market Street between the Embarcadero Station and Powell Street Station in advance of the driving of the two each Muni (upper) and BART (lower) tunnels. Although their capabilities were verified by testing, the schemes were actually never activated due to lack of ground movements, as the tunnels were located in the street, not directly below the buildings.

Finally, we have developed a third option of more substantial and permanent underpinning. This scheme will require extensive occupancy of ground or basement floors in existing buildings, and will be quite expensive.

However, if installed properly, it would protect existing structures against virtually any form of tunnel construction ground settlement, save for a catastrophic collapse.

It will involve extensive demolition, drilling, excavation, temporary support, grouting and concreting, and would need to be installed by a specialty contractor.

### **3.3 MICRO/MINI/PIN PILE UNDERPINNING**

We agree with NCC that the required bond stress between such a pile and existing weak foundation materials may well be excessive in many cases. Hence, these piles will have to be connected to new concrete grade beams and pile caps. In order for this underpinning to be effective, the piles would have to be founded in material below tunnel invert or below the assumed "zone of influence", where ground is not subject to movement.

Therefore, the founding of these piles must be in "certifiably" stable ground, as adjustments are difficult to make after piles are installed. This underpinning will also need to be installed below existing grades and slabs to allow affected portions of buildings to be restored and the



temporary excavations to be backfilled, in order that the various commercial enterprises can remain operational.

To that purpose, we have developed the following schemes of underpinning using small diameter drilled and grouted mini piles. These piles are generally formed by drilling a small diameter hole (say 8" to 10") to required depth. The hole is then filled with cement/water

grout under pressure and supplied with a high strength bar or strand tendon for vertical load capacity.

The application of this technique, as support of existing buildings, is discussed below for two general cases that represent the range of conditions likely to occur.

### **3.3.1 BUILDINGS WITH NUMEROUS INTERIOR COLUMNS:**

Clusters of these small diameter piles, ranging from a minimum of two to a maximum of about six, will be installed parallel to the tunnel alignment on each side of tunnel envelope at about 25-foot centers, as shown schematically on plate SK-3.

These pile clusters will be connected by a longitudinally running grade beam which will be reasonably continuous. We estimate this to be a reinforced concrete beam about 2'± wide and 4'± deep. Hence, there will be one such beam on each side of centerline of tunnel, spaced approximately 20' in each direction of centerline, for a transverse dimension of roughly 40' between such cap beams.

Wall footings and interior column footings will then be supported by a grillage of transverse beams as required, installed between the longitudinal grade beams described above. We envision these beams at 1' wide by 2' to 3' deep.

The overburden over top of tunnel varies from a low of 20'± to a high of about 45'. Accordingly, we estimate the length of the micro/mini piles to be about 80' long on the average. The longitudinal grade beams and the transverse pick-up beams will be installed in trenches excavated beneath the ground or basement slabs where such occurs, and then after connection to existing foundations is made, the beams are covered over, and original slab is restored. Plate SK-4 depicts this option.

This installation may prove difficult to perform in some buildings where occupancy of the ground slab is fully saturated.



Control underpinning would be less intrusive, but may require workers to enter facilities periodically to perform the maintenance jacking.

Preloadings of the support/underpinning system can easily be achieved prior to "covering" it over by post-tensioning grade beams.

Based on the soil/rock profiles and the overburden thickness, we are of the opinion that buildings north of Brannan Street will not need to be underpinned.

Based on the tunnel route eventually selected (short, medium or long radius tunnels), the underpinning requirements will vary substantially.

### **3.3.2 BUILDINGS WITH FEW INTERIOR COLUMNS**

We have now developed a second "site specific" underpinning scheme for the buildings affected by the "short" tunnel route which is shown on drawing SK-5.

This scheme varies from the "generic" scheme developed under item 3.3.1 above, in that it does not utilize the longitudinal grade beams due to the sparsity of interior columns. The grade beams are placed parallel to column rows and exterior wall footings. In this manner, we can reduce the grade beam installation and reduce the length of the connected pickup beams.

But again, similar to the scheme discussed under Section 3.3.1 (SK-4), the support piles are placed in two rows which parallel the tunnel on each side of centerline at 20' and 35', respectively.

### **3.4 ZONE OF INFLUENCE ON BUILDINGS AT SURFACE**

The width of the area at the ground surface which may be subject to deformation from tunnel driving has been estimated by us to be from 10' to 15' wider on each side than actual tunnel envelope. Hence, this zone of possible movement is envisioned to be from 65' to 75' wide, maximum.

Accordingly, if underpinning is to be installed, it needs to be extended beyond the limits of such a wide zone (70'± avg); see section on drawing SK-5.



We envision that this zone of possible disturbed ground mass rises from each bottom corner of excavation at a slope of 1H:2½V to top of tunnel, where it goes vertical to the ground surface. This is the zone which determines the extent of underpinning work to be undertaken within existing buildings.

### **3.5 ZONE OF "NO-LOAD" FOR U/P PILES**

The plane which defines the soil mass into which piles should be founded starts at the same bottom corners of excavation. From there it goes horizontal for 5' and then rises on a 1H:1V in each direction. We estimate that these small diameter, but pressure grouted piles will need to be extended about 20' below this plane on the average for a pile capacity of 100K±.

## **4. ESTIMATED COSTS**

### **4.1 UNDERPINNING COST ESTIMATE**

We have performed a couple of cost models for a few typical buildings, and our cursory analysis indicates that the unit cost of the two underpinning schemes can be calculated at \$1200± and \$2000± per linear foot of tunnel (max dimension) within the confines of the structure for control and micro/mini pile underpinning, respectively.

We assumed an average building covered 120LF of tunnel. We arrived at these costs in the following manner:



#### 4.1.1 CONTROL PIER UNDERPINNING & COLUMN PICK-UP

For this average building, we assumed we needed 25 control piers @ 3'x5'x 4' = 2.5 CY/pier.

We estimate the equivalent cost of all the work associated with the installation of the control piers @ \$500/CY. Hence, for each pier we estimated as follows:

Install - 2.5 CY x \$500	=	\$ 1,250.00
Monitor & Jack (3 wks)	=	750.00
Shim, grout, decommission	=	1,500.00
		<u><u>          </u></u>
TOTAL		\$ 3,500.00/pier

We assumed we had 8 column pick-ups, with the following unit costs:

Install at each column	=	\$ 1,500.00
Monitor & jack	=	1,000.00
Shim, grout, decommission	=	1,500.00
		<u><u>          </u></u>
TOTAL		\$ 4,000.00/column

Thus, 25 @ \$ 3,500.00	=	\$ 87,500.00
8 @ \$ 4,000.00	=	32,000.00
		<u><u>          </u></u>
TOTAL		\$120,000.00±

Thus, the cost per LF = \$120,000/120' = \$ 1,000/LF

Allow 20% for mark-up  
1.2 X \$1.0K = \$ 1,200/LF (of tunnel)



#### **4.1.2 MINI/MICRO PILE UNDERPINNING**

For the average 120'LF of tunnel building, we estimated:

60CY of grade beams @ \$750/CY	=	\$ 45,000.00
80CY of pick-up beams @ \$650/CY	=	52,000.00
50CY of misc. concrete @ \$500/CY	=	25,000.00
30 Mini piles @ \$2,500 as installed	=	75,000.00
		<hr/>
TOTAL		\$200,000.00

Final unit cost  $(200,000/120)1.2 = \$2,000/\text{LF}$

#### **4.2 TOTAL UNDERPINNING COST**

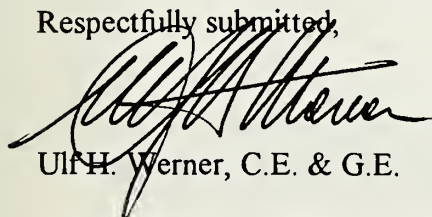
We have estimated the approximate linear footage for each of the three tunnel route alternatives at 500LF, 800LF, and 1250LF for short, medium and long, respectively.

Accordingly, the cost of underpinning can vary from a low of \$600,000 to a high of \$2,500,000 for the two extreme uses of control underpinning with the short radius tunnel route and micro/mini pile underpinning in combination with the long radius tunnel route, respectively.

**TABLE I - TOTAL UNDERPINNING COST**

<b><u>ROUTE</u></b>	<b><u>CONTROL TYPE UNDERPINNING</u></b>	<b><u>MICRO/MINI PILE UNDERPINNING</u></b>
Short Radius	\$ 600,000	\$1,000,000
Medium Radius	\$ 960,000	\$1,600,000
Long Radius	\$1,500,000	\$2,500,000

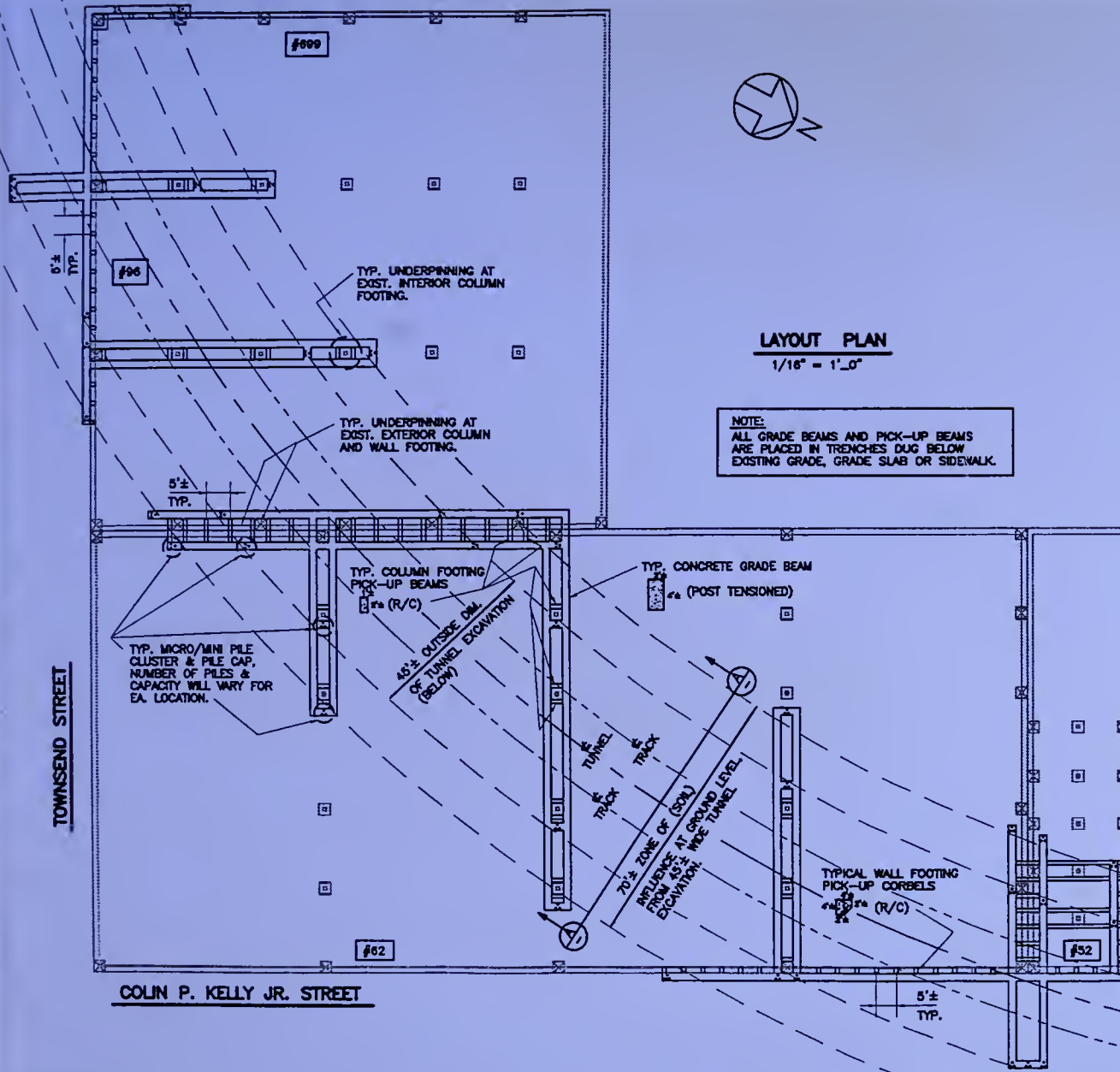
Respectfully submitted,



Ulf H. Werner, C.E. & G.E.



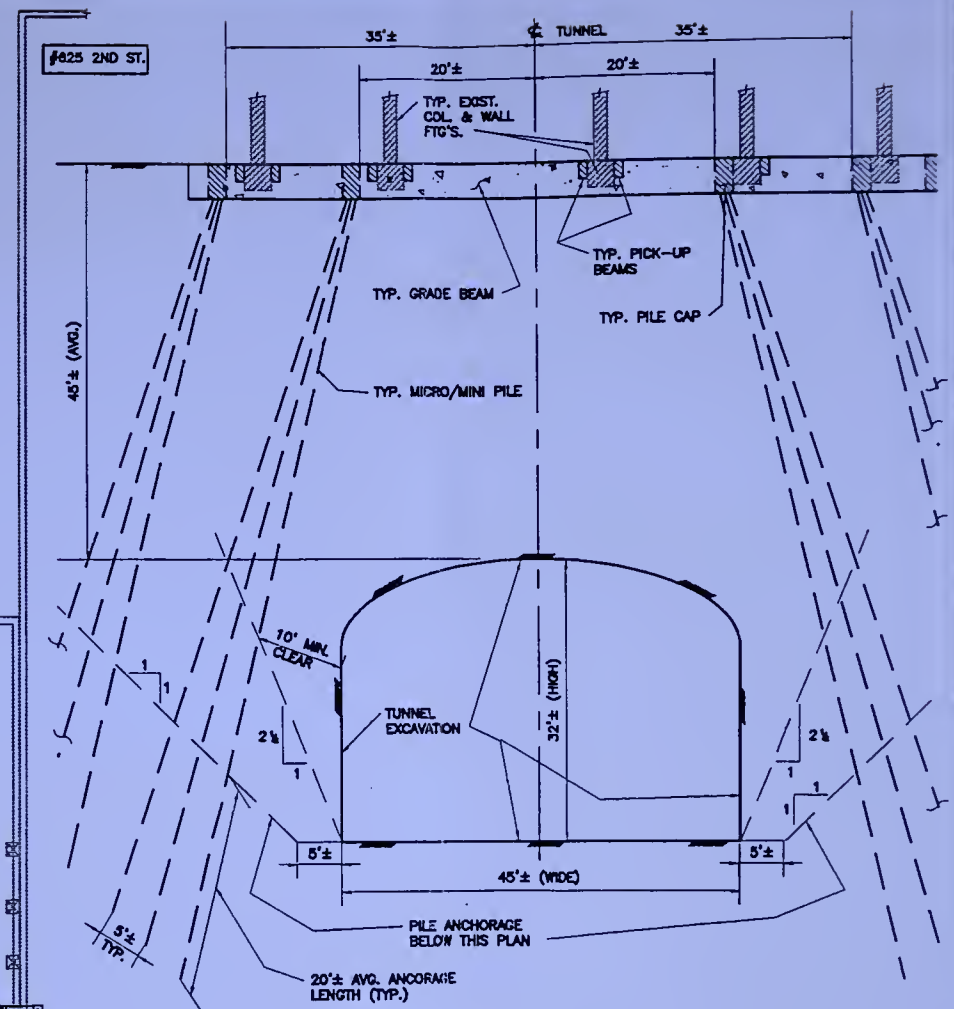
2ND STREET



LAYOUT PLAN




1/16" = 1'-0"

NOTE:  
ALL GRADE BEAMS AND PICK-UP BEAMS  
ARE PLACED IN TRENCHES DUG BELOW  
EXISTING GRADE, GRADE SLAB OR SIDEWALK.

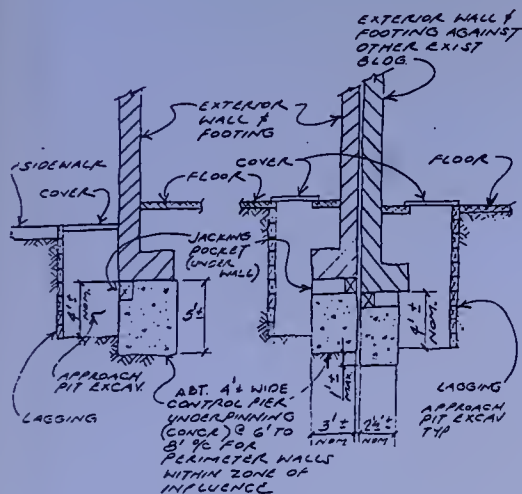


SECTION A

1/8" = 1'-0"

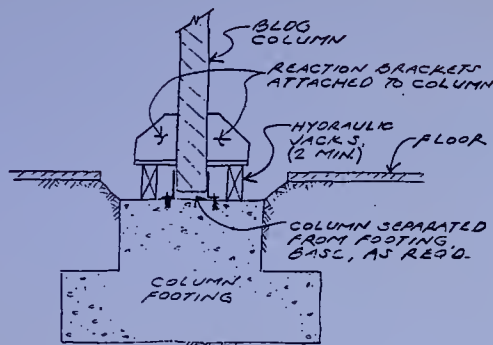
						DESIGNED BY UHW			ICF KAISER ENGINEERS, INC - DE LEUW CATHER & COMPANY SAN FRANCISCO CALIFORNIA	 WERNER AND ASSOCIATES 3001 SAN GABRIEL AVENUE, SUITE 200, SAN GABRIEL, CA 91764 TEL: 916/338-7400 FAX: 916/338-7408	CALTRAIN S.F. DOWNTOWN EXTENSION SAN FRANCISCO CALIFORNIA	CADD FILE NO DWG-101	CADD DATE 01MAY96		
						CHECKED BY CKY								SCALE AS NOTED	JOB NUMBER 95084
						DESIGNED BY TOB									
						CHECKED BY UHW									
						DATE 05MAY96									
REV	DATE	BY	SUB	APP	DESCRIPTION										





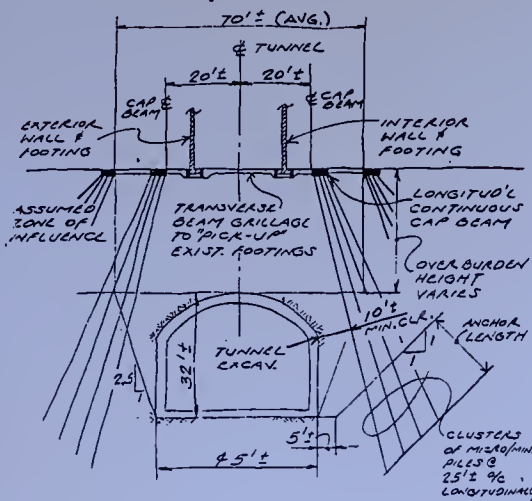
TYPICAL CONTROL PIER  
UNDERPINNING SECTIONS

SK-1



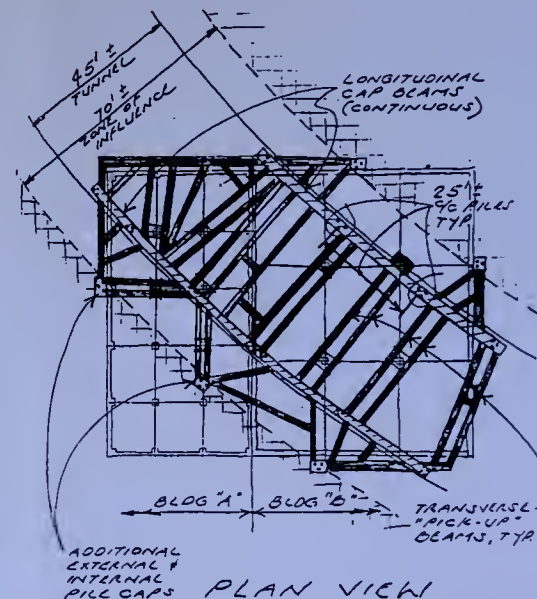
TYPICAL COLUMN  
PICK-UP SECTION

SK-2



TYPICAL MICRO/MINI PILE  
UNDERPINNING SECTION  
BETWEEN TOWNSEND AND  
BRANNAN STREETS  
(1" = 20'±)

SK-3



PLAN VIEW  
TYPICAL UNDERPINNING  
FOR MICRO/MINI PILE  
SCHEME

SK-4

CONTROL PIER & COLUMN PICK-UP  
SUPPORT SCHEME

GENERIC MICRO/MINI PILE UNDERPINNING  
FOR TYP. BUILDING W/ MANY INTERIOR COL'S.

DESIGNED BY  
UHW  
CHECKED BY  
CKY  
DATE  
10/6  
IN CHARGE  
UHW  
DATE  
06MAR96



ICF KAISER ENGINEERS, INC - DE LEUW CATHER & COMPANY  
SAN FRANCISCO CALIFORNIA



WERNER AND ASSOCIATES

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CALTRAIN S.F. DOWNTOWN EXTENSION  
SAN FRANCISCO CALIFORNIA

UNDERPINNING OF EXISTING HISTORIC BUILDINGS

TYP. CONTR. PIERS, COL. PICK-UP & PILE U/P

CADD FILE NO  
DWG-101

CADD DATE  
01MAR96

SCALE  
AS NOTED

JOB NUMBER  
95084

DRAWING NO  
SK-1,2,3,4

REV  
A













